State of California
The Resources Agency
Department of Water Resources
DIVISION OF DESIGN AND CONSTRUCTION

SEISMIC STABILITY EVALUATION

of the

SACRAMENTO-SAN JOAQUIN DELTA LEVEES

VOLUME I

PHASE I REPORT: PRELIMINARY EVALUATIONS AND
REVIEW OF PREVIOUS STUDIES

August 1992

C-072219

FOREWORD

The United States Geological Survey recently concluded that a large magnitude earthquake has a high probability of occurring in the San Francisco Bay area within the next 30 years. This conclusion and the occurrence of the Loma Prieta Earthquake in 1989 has intensified concerns relating to the stability of levees in the Sacramento-San Joaquin Delta.

This report provides preliminary assessments of the susceptibility of Delta levees to damage from future earthquakes and an evaluation of the opportunity for that damage to occur. Also, this report provides information that can be used to identify future phases of work needed to develop data that does not presently exist, particularly on the behavior of organic soils during earthquakes.

This work was performed with guidance from a Board of Consultants established by the Department. This board consists of three experts in the fields of seismology, earthquake engineering, and geotechnical engineering.

It is intended that the work presented in this report be followed by two additional phases of investigation. The additional phases of study would better establish engineering properties of the organic soils common in the Delta so that improved assessments relating to seismic stability can be made. With improved assessments, design criteria can be established, and a rational approach can be pursued in the management of existing and future Delta facilities.

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SEISMIC STABILITY EVALUATION OF THE SACRAMENTO-SAN JOAQUIN DELTA LEVEES

DIVISION OF DESIGN AND CONSTRUCTION
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This report has been prepared under my direction as the professional engineer in direct responsible charge of the work, in accordance with the provisions of the Professional Engineers' Act of the State of California.

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SEISMIC STABILITY OF DELTA LEVEES

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1. EXECUTIVE SUMMARY

1.0 INTRODUCTION

Concern for the seismic stability of levees protecting islands in the Sacramento-San Joaquin Delta has increased since the occurrence of the 1989 Loma Prieta Earthquake. Recent assessments by the United States Geological Survey indicate that there exists a high probability that a large magnitude earthquake will occur in the San Francisco Bay area within the next 30 years. These assessments have heightened concerns related to the seismic stability of Delta levees.

Concerns exist because the islands are commonly 10 to 15 feet below sea level and the levees are composed generally of uncompacted, sands and silts, built without engineering design and/or good construction methods. Levees composed of such materials commonly experience liquefaction and damage during moderate to strong earthquake shaking. During periods of low Delta outflow, the consequence of inundating any island in the Delta would be to draw saline water from the San Francisco Bay into the Delta. This would immediately limit or stop the export of water until the island could be reclaimed. For any one of the eight western Delta islands, permanent inundation would result in significant degradation of water quality.

1.1 SCOPE OF INVESTIGATION

The purpose of the investigations outlined in this report is to produce a preliminary assessment of the stability of Delta levees during future earthquake shaking. This assessment is based on the present condition of the levee system. It was recognized that making meaningful assessments regarding Delta levees would be challenging due to the extensive lengths of levees involved, over six hundred miles, and the large variability in levee geometries and material properties. It was also recognized that relatively little was known concerning the dynamic response and strength characteristics of Delta organic soils during earthquake shaking. In order to produce reliable assessments, the approach adopted was to develop tasks for three phases of investigation:

Phase I

1. Review previous studies and historical data relating to the seismic stability of Delta levees.

2. Perform preliminary studies to estimate bedrock ground motions using deterministic and probabilistic methods.

3. Perform preliminary dynamic response analyses to investigate the amplification/attenuation characteristics of Delta soil profiles.

4. Perform preliminary evaluations of liquefaction potential and estimates of earthquake-induced deformations.

5. Develop work programs for Phases II and III.

6. Produce Phase I report.

Phase II

1. Perform field and laboratory geotechnical studies and sponsor research on the dynamic response characteristics of organic soils.

2. Install surface and subsurface strong motion instruments at three or more locations in the Delta.

3. Produce Phase II report.

Phase III

1. Continue laboratory studies and perform additional dynamic response analyses. Refine seismic stability evaluations of Delta levees.

2. Produce Phase III report.

Regardless of the results developed in these investigations, it is not the intention to either repair existing or design new levees to meet standards developed for earth dams. Rather, it is the purpose of these studies to develop information as to the susceptibility and opportunity for Delta levees to sustain damage during earthquakes. With this information, the degree of risk can be estimated, design criteria can be established, and a rational approach can be pursued in the management of existing and future Delta facilities.

This report presents the results of the Phase I investigations.

C-072237

1.2 SUMMARY OF FINDINGS

1. Since the reclamation of the Delta islands began in the late 1860s, the bedrock and stiff soil lying below the soft organic soils common throughout the Delta have never been subjected to significant earthquake-induced ground motions. Damage intensity maps indicate that rock and stiff soil sites at the periphery of the Delta have experienced peak accelerations no higher than about 0.1g to 0.15g. Within the central portions of the Delta, outcrops of rock or stiff soil would have experienced corresponding peak accelerations no higher than about 0.1g if such "stiff" site conditions had been present in this region.

2. No levees are known to have failed in the Delta due to earthquake shaking. The most significant confirmed damage in the Delta due to earthquake shaking appears to be the approximate three feet of settlement reported for a Santa Fe railroad bridge at the Middle River crossing during the 1906 San Francisco earthquake. Since that time, levee heights have increased by approximately 100 percent.

3. The limited damage to Delta levees reported by Finch (1985) due to earthquakes occurring between 1979 and 1984 is in most cases difficult to definitively attribute to earthquake shaking. According to re-interviews of witnesses, there was often pre-earthquake distress at most of the sites mentioned in the report. In addition, some of the damage reported may be related to other factors (e.g., ongoing levee subsidence or levee modifications being made at the time of the earthquake). Further, even those reports which were verified do not indicate a level of damage significantly above that which existed prior to the earthquake. Consequently, these incidents do not reveal significant information on the relative vulnerability of Delta levees to seismic shaking.

4. There was significant damage to a Southern Pacific railroad embankment on soft soil west of the Delta in the Suisun Marsh during the 1906 San Francisco earthquake. The embankment settled several feet for a significant length (some reports indicate as much as 3 to 6 feet for over 1,000 feet). This location had experienced distress the previous year and the earthquake-induced damage appears to be related to bearing failure rather than liquefaction. An outcrop of rock or stiff soil at this

location would have been expected to have experienced peak ground accelerations of about 0.18g during the Magnitude 8+ event on the San Andreas Fault.

5. Several assessments regarding the risk of levee failure during earthquake shaking have been previously performed. Some of these studies have been performed for specific projects (e.g., Mokelumne Aqueduct) while others have been for the general Delta region. The general conclusion from these studies is that levees and/or their foundations would liquefy and result in island inundation if surface motions exceeded some critical acceleration, generally reported to be between 0.1g to 0.2g.

6. Probabilistic methods used in the current study indicate that, if the western and southwestern Delta were composed of stiff soil or rock outcrops, these areas would have a 50 percent probability of non-exceedance for peak accelerations of approximately 0.15g within a 30-year exposure period. Because there are significant thicknesses of soft organic soils in these areas, it is unclear whether such ground motions would be either amplified or attenuated through the peaty deposits.

7. If the postulated Coast Range-Sierra Nevada Fault Zone ruptured beneath the Delta with a Magnitude 6.5 earthquake, then central portions of the Delta could experience peak accelerations greater than 0.4g if these areas were composed of stiff or deep, cohesionless soils. Again, because there are significant thicknesses of soft organic soils in these areas, it is unclear what proportion of these large base motions would be propagated upward. However, even if ground motions were strongly attenuated through the peaty soils, the base motions are so high that the resulting motions within the levees near the surface would still be expected to be more than twice the levels of motion previously experienced in the Delta.

8. Several areas in San Francisco and Oakland along the periphery of the San Francisco Bay experienced significant ground motion amplification during the 1989 Loma Prieta earthquake due to the presence of deep deposits of cohesive soil. This amplification resulted in significant amounts of liquefaction of bay fills and damage to structures. However, locations in the southwest portion of the Delta (e.g., Clifton Court, Victoria Island, and Byron Tract) are about the same distance from the Loma Prieta epicenter as were these Bay

Area sites (about 60 miles). No significant damage was reported in the Delta areas. Consequently, this result would suggest that soils in these areas do not have the same amount of amplification characteristics as do the bay margin soil profiles.

9. Preliminary dynamic response analyses performed in this study show that strain-dependent properties assigned to peat layers greatly affect the ground motions propagated through to the levee fills. If the peat layers are assigned properties similar to those developed for Union Bay, Washington, peats, the surface motions commonly have about 1/2 to 3/4 of the peak accelerations input at the base. If the peat layers are assigned properties similar to those for San Francisco Bay Mud, then the motions are commonly 20-60 percent higher and up to 100 percent higher than the peak accelerations input at the base. However, there is considerable uncertainty regarding which set of properties properly represents the behavior of peaty soils. Due to the large variability in soil profiles within the Delta region, there would be a range in amplification characteristics expected.

10. The general level of data available to characterize the liquefaction potential of levee and foundation soils is limited both in quantity and quality. Cursory examination of SPT data available from recent studies at Sherman Island and other sites indicate that the cyclic stress ratio required to trigger liquefaction in many levees during a magnitude 7.5 earthquake would generally be expected to be between 0.13 and 0.18. Peak accelerations sustained at the surface of many levees in the range of 0.15g to 0.2g would be expected to trigger extensive liquefaction and result in significant damage to the levee in the form of cracking and slumping. Levee failure could be expected for reaches where levee freeboard and/or cross section is limited. No formal assessments were made for foundation soils, as the available data suggests a lack of continuity for many sites and such assessments would simply reflect the severity of the assumptions made.

11. There are several unknowns which significantly influence evaluations of levee stability during earthquake shaking. The unknowns which have the largest effects on assessments of levee stability during earthquakes are listed below in descending order of importance:

a. Amplification/attenuation characteristics of shallow organic soils.

b. Liquefaction resistance of levee fills.

c. Strength loss potential in cohesive/organic soils following earthquake shaking.

d. Amplification/attenuation characteristics of deep soil profiles.

e. Liquefaction resistance of foundation soils.

f. Probability of Coast Range-Sierra Nevada Fault Zone producing a large magnitude earthquake ($M \geq 6.5$) within the Delta.

1.3 PRELIMINARY CONCLUSIONS

1. Levees with significant thicknesses of peaty soils cannot be reliably assessed for their opportunity to be damaged during earthquake shaking due to the great uncertainty regarding the strain-dependent properties of such soils to either amplify or attenuate ground motions. Nevertheless, the preliminary studies completed in this evaluation suggest that the amplification factors (ratios of peak levee crown accelerations to bedrock peak accelerations) may commonly range between 1 and 1.6. Given an assumed amplification factor of unity and a 30-year exposure period with a 50 percent probability of non-exceedance for bedrock accelerations, only the western portion of the Delta would be expected to have moderately high susceptibilities for levee damage (see Figure 1-1). However, for the same bedrock accelerations but with an assumed amplification factor of 1.6, both the western and central portions of the Delta would have moderately high susceptibilities for levee damage (see Figure 1-2). These two plots describe our current perception of the probable range in susceptibility for a 30-year exposure period. For higher exposure periods, the expected susceptibilities for levee damage and failure significantly increase (see Chapter 8).

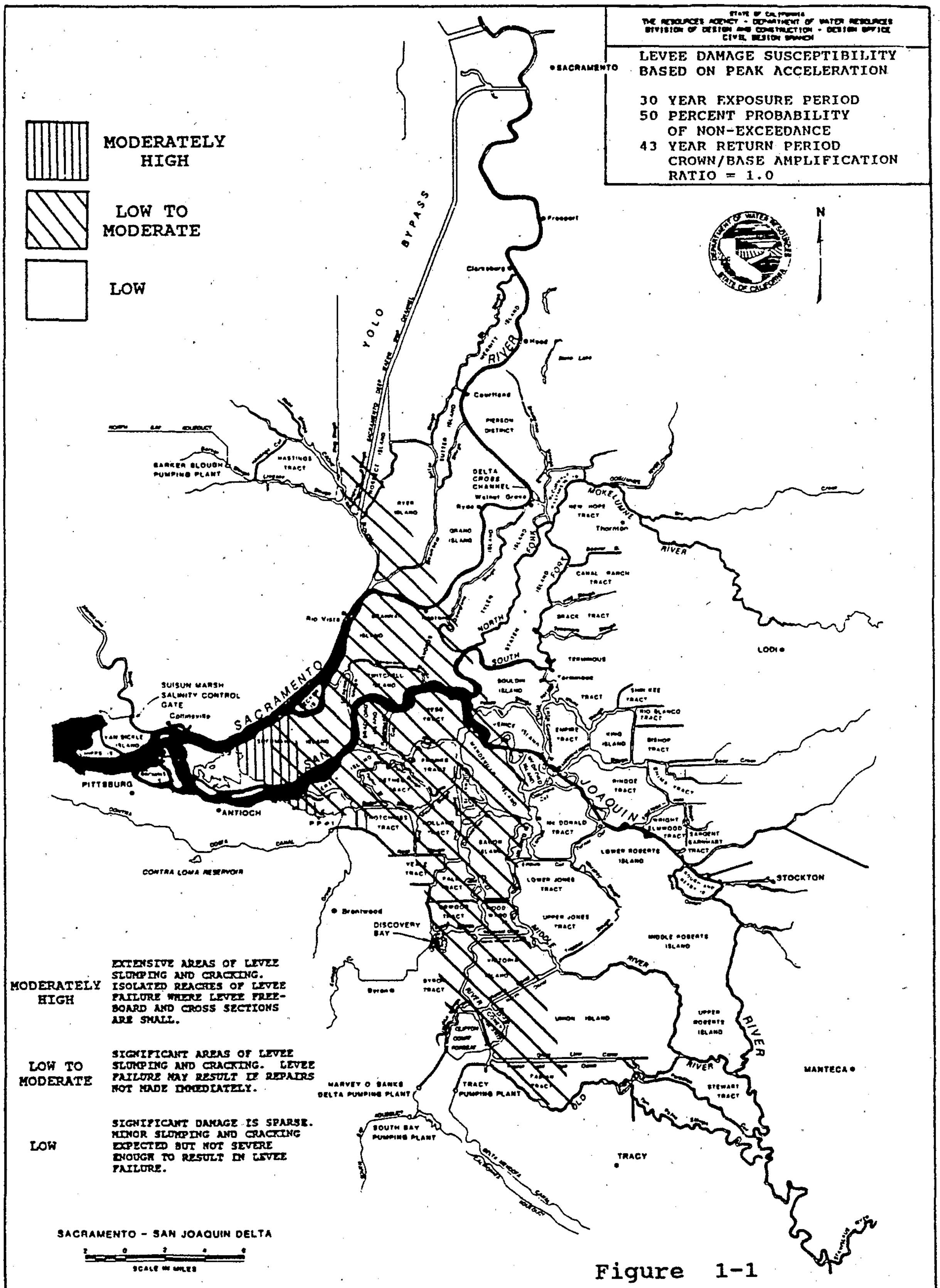
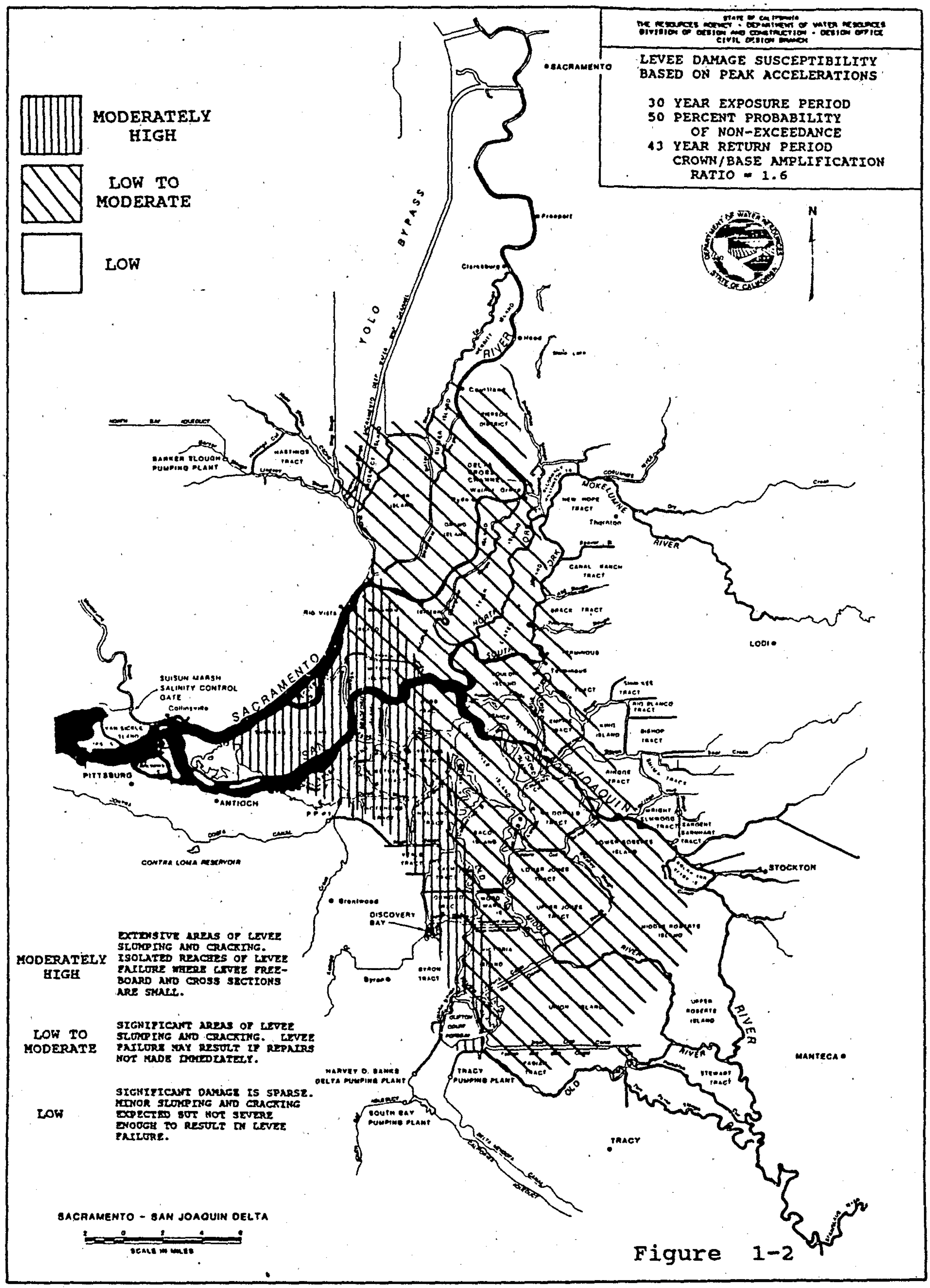

LEVEE DAMAGE SUSCEPTIBILITY
BASED ON PEAK ACCELERATION
30 YEAR EXPOSURE PERIOD
50 PERCENT PROBABILITY
OF NON-EXCEEDANCE
43 YEAR RETURN PERIOD
CROWN/BASE AMPLIFICATION
RATIO = 1.0
MODERATELY HIGH
LOW TO MODERATE
LOW
N
SACRAMENTO
YOLO BYPASS
RIVER
SACRAMENTO
SAN
JOAQUIN
RIVER
MOKELUMNE RIVER
NORTH
SOUTH
MIDDLE
OLD
RIVER
LODI
STOCKTON
MANTECA
TRACY
PITTSBURG
ANTIOCH
DISCOVERY BAY
SUISUN MARSH SALINITY CONTROL GATE
BARKER SLOUGH PUMPING PLANT
DELTA CROSS CHANNEL
CONTRA LOMA RESERVOIR
HARVEY O BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
SOUTH BAY PUMPING PLANT
UNION ISLAND
LOWER JONES TRACT
UPPER JONES TRACT
MIDDLE ROBERTS ISLAND
UPPER ROBERTS ISLAND
LOWER ROBERTS ISLAND
MODERATELY HIGH
EXTENSIVE AREAS OF LEVEE SLUMPING AND CRACKING. ISOLATED REACHES OF LEVEE FAILURE WHERE LEVEE FREEBOARD AND CROSS SECTIONS ARE SMALL.
LOW TO MODERATE
SIGNIFICANT AREAS OF LEVEE SLUMPING AND CRACKING. LEVEE FAILURE MAY RESULT IF REPAIRS NOT MADE IMMEDIATELY.
LOW
SIGNIFICANT DAMAGE IS SPARSE. MINOR SLUMPING AND CRACKING EXPECTED BUT NOT SEVERE ENOUGH TO RESULT IN LEVEE FAILURE.
SACRAMENTO - SAN JOAQUIN DELTA
SCALE IN MILES

Figure 1-1


STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
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SCALE IN MILES

Figure 1-2

2. Sandy and/or silty levees that are founded on soil profiles without significant thicknesses of peaty soils, such as previous levee failure sites where the organic soils have been scoured out or old stream channels infilled with sand, do not possess soils which could significantly attenuate or dampen earthquake motions. Consequently, there is a relatively high degree of certainty that such sites will have amplification factors equal to at least unity. Thus, for bedrock accelerations that would be sustained for a 30-year exposure period with a 50 percent probability of non-exceedance, the susceptibilities for levee damage at such sites would be comparable for those shown in Figure 1-1, or higher.

3. If Delta soil profiles exhibited the same amplification characteristics as soil profiles along the margin of the San Francisco Bay, the outlook for Delta levees all throughout the Delta during moderate to large earthquake shaking would be grave. However, the limited data available do not support this because Delta sites along the southwest periphery have not experienced significant damage during recent earthquakes. Levees along Clifton Court, Victoria Island, and Byron Tract were the same distance from the 1989 Loma Prieta epicenter as was San Francisco, but no significant damage was observed at these levees. Accordingly, this may suggest that soil profiles in these portions of the Delta are either unlikely to exhibit such amplification abilities, or that the Delta levees are significantly stronger than generally considered.

1.4 RECOMMENDATIONS FOR FUTURE PHASES OF WORK

The studies performed in previous reports and in the current evaluation suffered from the fact that the Delta is a very large environment with tremendous variability in levee geometries and properties. In addition, there are a great many unknowns regarding the dynamic properties of the peaty foundation layers which commonly exist beneath the levee system. Future phases of study should attempt to reduce some of the major uncertainties by performing the following:

1. Install strong-motion accelerometers at three to four levee sites in the Delta. Instruments should be installed both at the surface and at several subsurface depths so that earthquake amplification through the soft organic soils can be measured.

2. A geologic model for the deeper soils deposits should be developed for the Delta. If possible, it would be desirous to develop depth contours of soils sufficiently consolidated to be treated as rocklike in amplification characteristics.

3. Field and laboratory testing should be performed to better determine the static and dynamic properties of organic soils.

4. Field and laboratory testing should be performed to better determine the liquefaction potential of levee and foundation soils.

5. Investigate the potential activity of the Coast Range-Sierra Nevada Boundary Zone.

6. Repeat the analyses and calculations performed in this study with the improved data that has been obtained.

2. GEOLOGIC AND HISTORICAL SETTING

2.0 GEOLOGY OF THE DELTA

The Sacramento-San Joaquin Delta, located at the confluence of the Sacramento and San Joaquin Rivers, is a unique feature of the California landscape (see Figure 2-1). The Delta is part of the Central Valley geomorphic province, a northwest-trending structural basin separating the primarily granitic rock of the Sierra Nevada from the primarily Franciscan Formation rock of the California Coastal Ranges (CWDD, 1980). The basin is filled with sedimentary deposits as depicted in Figure 2-2.

Table 2-1 presents the geologic formations that are present in the Delta area (McClure, 1956). The Delta occurs in an area that contains 5 to 10 km of sedimentary deposits, most of which accumulated in a marine environment about 175 million years ago to 25 million years ago. During the Cretaceous and Tertiary Periods, this area was a structural basin that received thick accumulations of sediment from the Sierra Nevada to the east and from the emerging Coast Range to the west. Uplift of the Coastal Ranges was accompanied by tilting and partial erosion of the older, previously deposited and consolidated sedimentary rock strata.

Since late Quaternary time, this area has undergone several cycles of deposition, non-deposition, and erosion, resulting in the accumulation of a few hundred feet of poorly consolidated to unconsolidated sediments overlying the Cretaceous and Tertiary formations. Fluctuating climatic cycles caused an exchange of water between the ocean and large continental ice sheets, resulting in the rise and fall of sea level relative to the land. Delta peats and organic soils began to form about 11,000 years ago during one of the rises in sea level (Shelmon and Begg, 1975). This rise in sea level created tule marshes that covered most of the delta. Peat formed from repeated burial of the tules and other vegetation growing in the marshes.

During the cycles of erosion and deposition, streams were entering from the north, northeast, and southeast. These included the Sacramento, Mokelumne, and San Joaquin Rivers. As the rivers merged, they formed a complex pattern of islands and interconnecting sloughs. River and slough channels were repeatedly incised and backfilled with sediments with each major fluctuation. These processes were complicated by concurrent subsidence and tectonic changes in land surface.

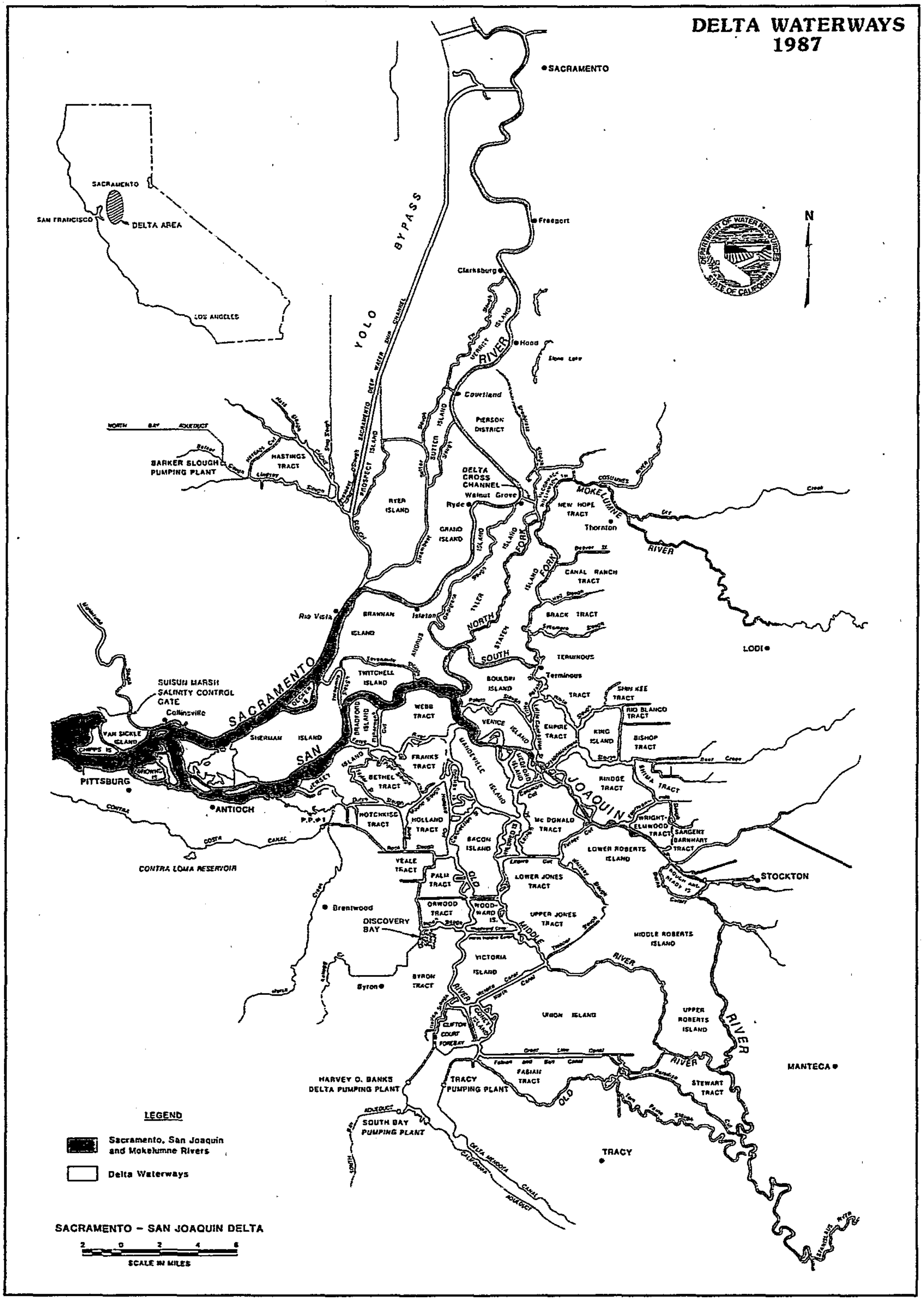

Figure 2-1: Delta Waterways, (From DWR, 1987)

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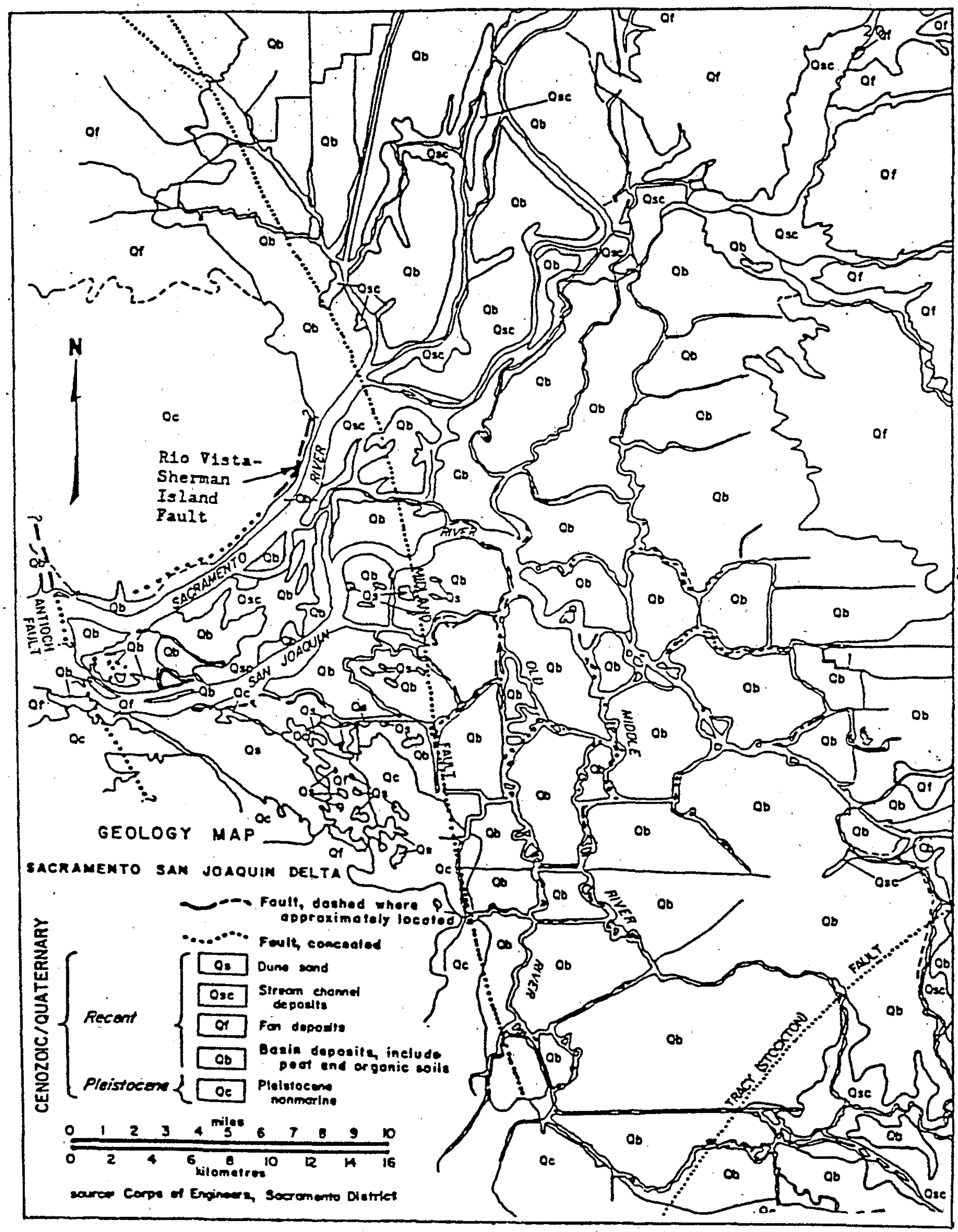


Figure 2-2: Geology of the Delta

Table 2-1: Geologic Formations, Sacramento Area (after McClure, 1956)

Geologic Age			Formation and Symbol	Approximate Thickness	Physical Characteristics
Quaternary	Recent	Sacramento-San Joaquin Delta Deposits*	Sacramento-San Joaquin Delta Deposits (Qs, Qac, Qo)	100 feet	Predominately impervious clays and silts, with some sand. Becomes highly organic in central Delta area.
Quaternary	Recent / Pleistocene	Sacramento-San Joaquin Delta Deposits*	Victor Formation and Related Continental Sediments (Qal, Qf)	150 feet	Unconsolidated gravel, sand, silt, and clay deposits, with extensive sand and gravel stringers.
Quaternary	Pleistocene	Sacramento-San Joaquin Delta Deposits*	Arroyo Seco Gravel and Unnamed Pleistocene Gravels	150 feet	Sand, gravel, silt, and clay.
Quaternary	Pleistocene	Sacramento-San Joaquin Delta Deposits*	Older Alluvium (Qoal)	Unknown	Sand, gravel, silt, and clay.
Quaternary / Tertiary	Pleistocene / Pliocene	Sacramento-San Joaquin Delta Deposits*	Montezuma Formation, Tehama Formation, Laguna Formation, Tulare Formation (TQc)	500 to 1,000 feet	Poorly sorted sand, gravel, silt, and clay.
Tertiary	Miocene		Mehrten Formation	400 feet	Conglomerate, silt, and clay, with interbedded lenses of black sands and agglomeratic material derived from andesitic mudflows.
Tertiary	Miocene		Valley Springs Formation	500 feet	Consolidated rhyolitic tuffs, conglomerates, clay-shales, and sandstones.
Tertiary	Miocene		San Pablo Group	1,000 feet	Interbedded massive sandstones and shale of continental and marine origin. Contemporaneous in part with the Mehrten and Valley Springs formations.
Tertiary	Eocene		Markley Formation and Undifferentiated Lower Eocene Formations	Unknown	Marine sands and shales.
Cretaceous			Moreno Formation, Panoche Formation, Chico Formation	Unknown	Marine sands and shales.
Pre-Cretaceous			Pre-Cretaceous Consolidated Rock	Unknown	Igneous and metamorphic rocks.

* The delta deposits comprise impervious clays and silts, with some sand and gravel.

Debris produced by hydraulic mining during the gold rush of the mid-1800s disrupted the natural depositional history of the Delta. Hundreds of thousands of tons of silt were washed from the Sierra Nevada into the Delta. This debris filled stream channels, caused flooding, and raised the natural levees along Delta streams and sloughs.

Continuing geologic processes in the Delta include:

- Erosion and deposition in rivers and stream channels.
- Tectonic subsidence.
- Diagenesis of deeper sediments.
- Scouring and deposition in islands due to levee breaks.
- Ponding of water in the islands.
- Subsidence due to deflation and oxidation of peats.

Geologic deposits of the Delta include:

- Peat and organic soils.
- Floodplain deposits.
- Stream channel deposits.
- Alluvial fan deposits.
- Pleistocene nonmarine deposits.
- Lithified marine deposits at depth.

2.1 PEAT AND ORGANIC SOILS

Classification of peat for engineering purposes has not been standardized. Peat is defined by the American Society for Testing and Materials (1992) as follows:

> "A naturally occurring highly organic substance derived primarily from plant materials. Peat is distinguished from other organic soil materials by its lower ash content (less than 25 percent by dry weight) and from other phytogenic material of higher rank (that is, lignite coal) by its lower calorific value on a water saturated basis."

This standard classification of peat has been developed mainly for agricultural/horticultural uses so that the peat-producer can better identify the product and the peat-consumer better select the materials to meet requirements.

The term peat as used in geotechnical engineering refers to highly organic soils and is used to describe soils that appear composed of organic matter, dark in color, and having an organic odor. Even organic soils containing more than 25 percent ash content are classified as peat by some geotechnical engineers. Unfortunately, this classification method can group a wide range of soils which can behave very differently under particular loading conditions (e.g., earthquake loading) under one generic term -- peat.

Many properties of organic soils are not well known. Particularly, little attention has been given their dynamic properties. Knowledge of the dynamic behavior of the organic soils in the Delta is essential for the determination of ground response to earthquake shaking.

As mentioned previously, Delta peat and organic soil began to form about 11,000 years ago during one of the rises in sea level. The effect of the high water level was twofold; the water supported marsh plant life and it prevented or greatly retarded the natural decomposition of plant life that would normally occur through oxidation. The lack of oxidation enabled vegetation to accumulate and form layers of peat. Figure 2-3 is an organic isopach map of the Delta and shows the approximate thickness of organic soils in the Delta. The map was developed by Allsup and Dudley (DWR, 1976), and is based primarily on data obtained along or near the levee system. Due to the large variability in the depositional environment, the map represents only a very general indication of the thickness of organic soils that may be present at a particular location.

2.2 REGIONAL FAULTING

The Sacramento-San Joaquin Delta lies in a seismically active region (see Figures 2-4 and 2-5). According to a recent U. S. Bureau of Reclamation report (Ake, et al., 1991), most of the late Quaternary faults in the Central California Coast Range region and near the Delta can be considered part of the San Andreas Fault system. The San Andreas Fault system refers to the network of faults with predominantly right-lateral strike slip that collectively accommodate most of the relative motion between the North American and Pacific plates (Wallace, 1990).

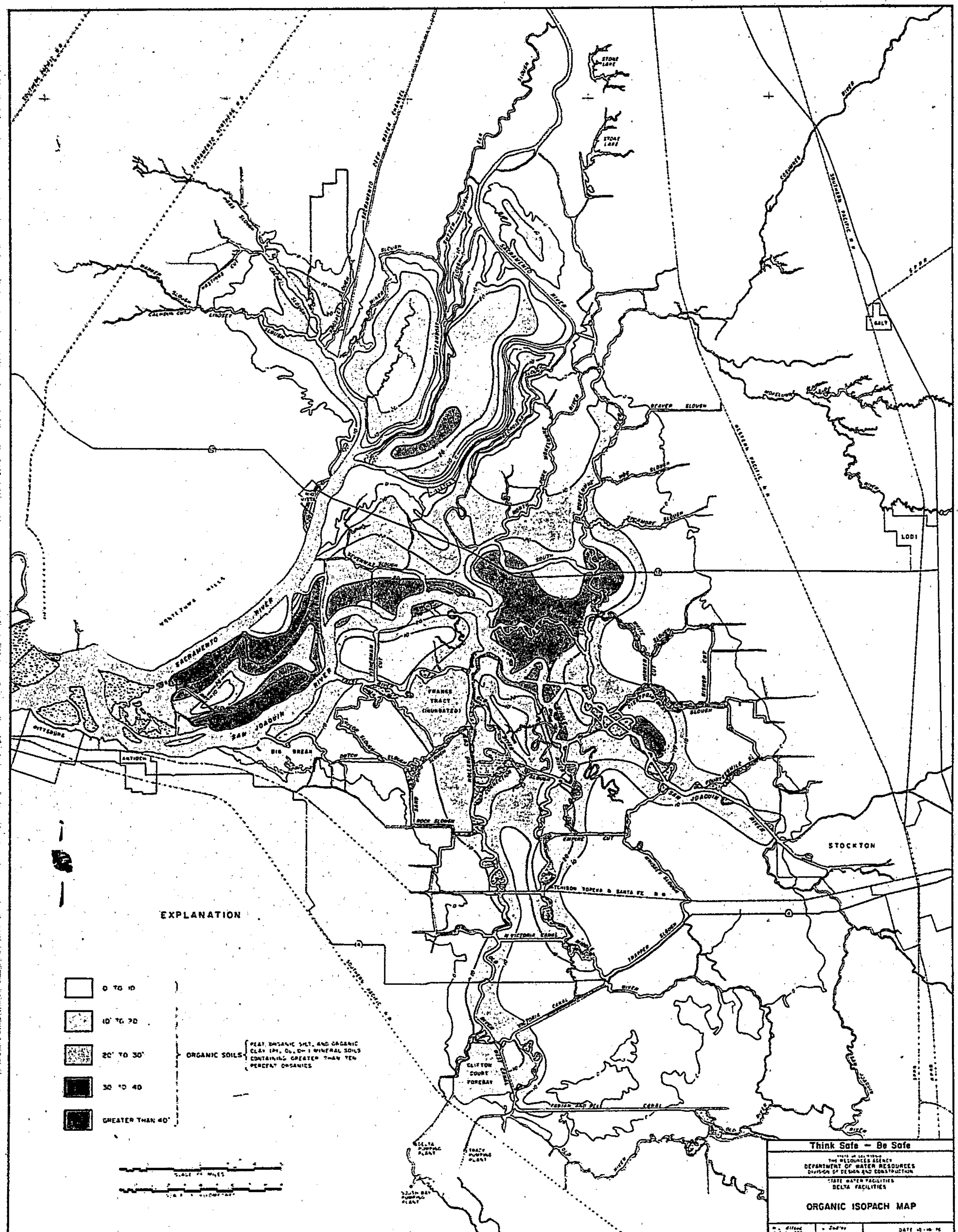


Figure 2-3: Organic Isopach Map of the Delta, (From DWR, 1976)

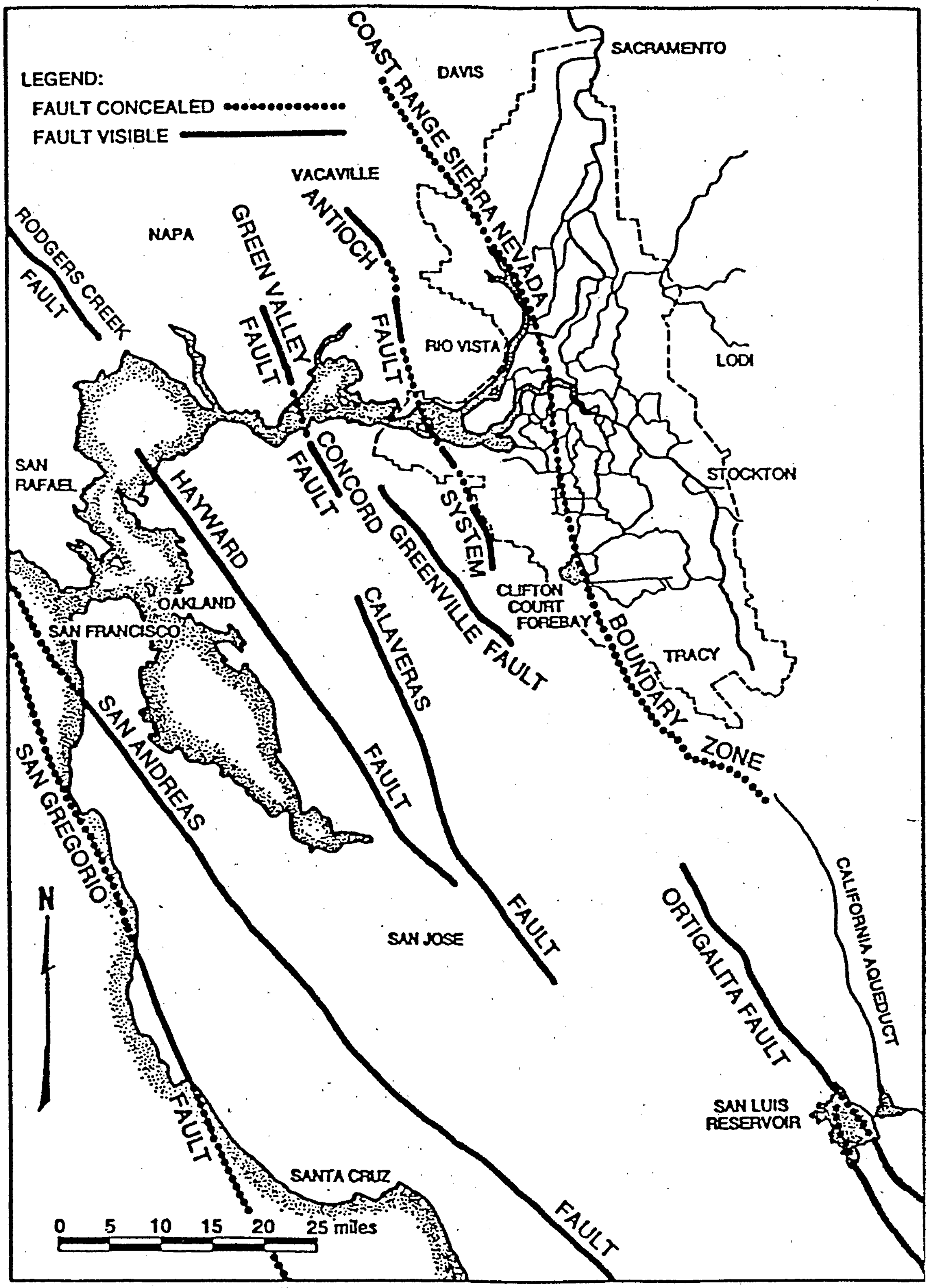


Figure 2-4: Regional Fault Sources

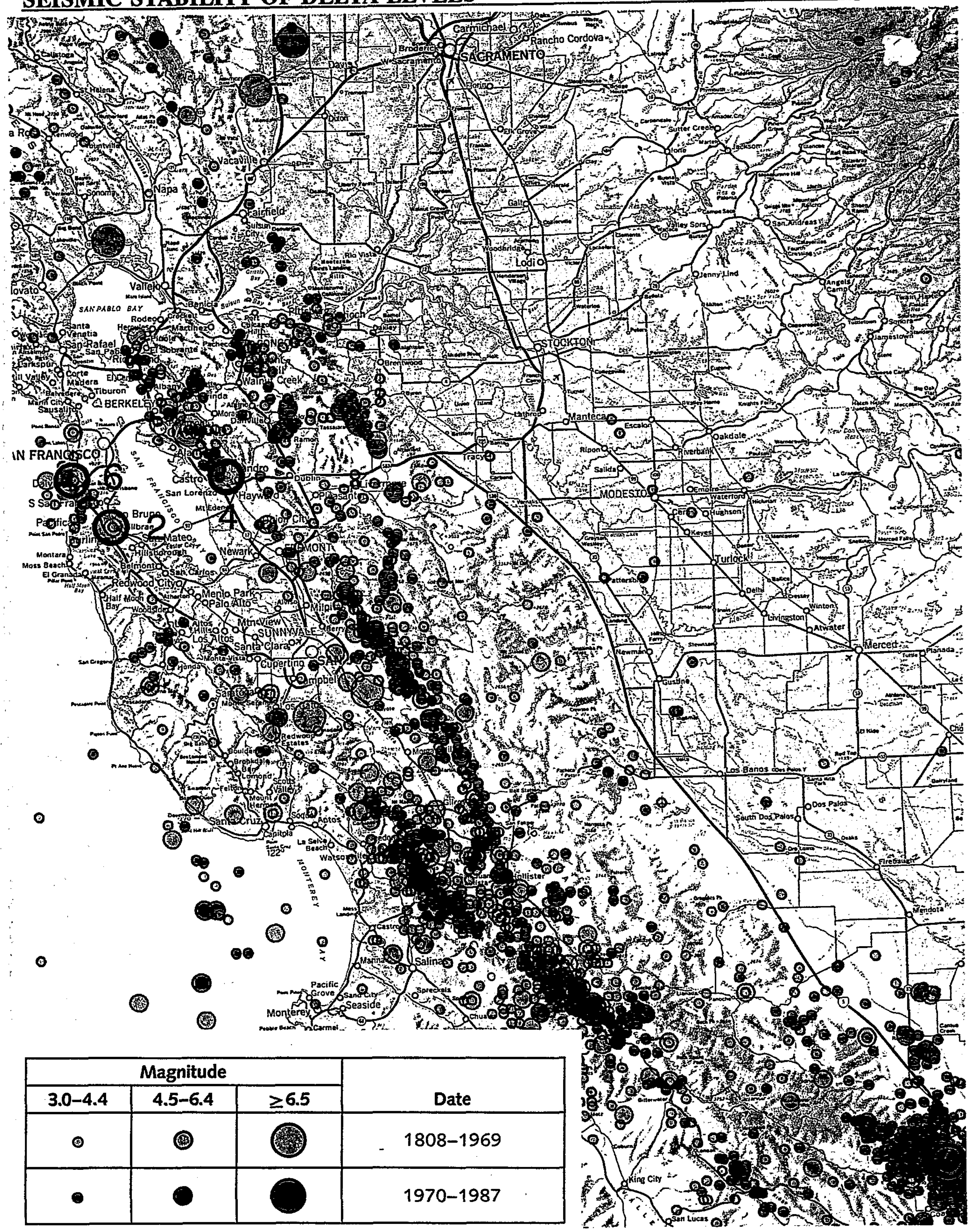


Figure 2-5: Regional Seismicity (From USGS, 1987)

The physiographic boundary between the Coast Ranges and the Great Valley also appears to represent a fundamental tectonic boundary. Ake, et al. (1991), states:

> "The western boundary zone between the Sierran Block and the Coast Ranges is thought to be a compressional boundary characterized by a zone of thrust faulting, reverse faults, and folding."

The Delta region lies astride this boundary zone.

The following sections contain brief descriptions of each fault that is expected to impact the seismicity of the Delta, and a summary of fault characteristics that were input to determine estimates of peak bedrock acceleration using both the deterministic and probabilistic approaches presented in Chapter 4.

San Andreas Fault

Ake, et al. (1991), assign a characteristic magnitude of M = 8 to this best studied fault of all those in the region. The fault is the site of an historic M = 8.25 earthquake in 1906. A 1988 USGS study is cited as a reference. A more recent USGS analysis of fault behavior suggests a higher slip rate of 19 mm/yr (USGS, 1990) than the 16 mm/yr shown in their 1988 paper. This higher rate was used in the current study.

The San Andreas Fault is treated in this study as a single, 420-km-long segment with endpoints at San Juan Bautista in San Benito County and Shelter Cove in Humboldt County, essentially combining the North Coast and San Francisco Peninsula segments as outlined by USGS (1988). Based on the earthquake recurrence relationship of the San Andreas Fault, two "b" values (Idriss, 1989) are used as input for probabilistic studies (see Chapter 4); b1, the steeper slope, occurs between the minimum magnitude of 4.5 and the transition magnitude of 6.5; b2, the gentler slope, occurs between M = 6.5 and the maximum magnitude earthquake of 8.5.

San Gregorio Fault

The San Gregorio Fault is the northernmost of a 400-km-long set of coastal faults lying southwest of the main trace of the San Andreas. The San Gregorio Fault extends northward from Monterey Bay, joining the San Andreas about 20 km northwest of San Francisco, near Bolinas Bay. Where it cuts the coastline near Point Ano Nuevo, the San Gregorio Fault is a complex three-to-five km wide zone of near-vertical strike-slip and northeast-dipping reverse faults (Wallace, 1990). This fault, which is considered capable of generating a 7.5 magnitude earthquake, was included in the deterministic analysis, but because of its distance from the Delta (55 miles west of Sherman Island) it was not considered as a potential seismic source in the probabilistic analysis.

Hayward Fault

Ake, et al. (1991), assign a characteristic magnitude of M = 7 to this fault. The 1988 USGS study is cited as a reference. Historic earthquakes include two M = 6.8 events, in 1868 and 1836 (Toppozada and others, 1981). The recently revised slip rate of 9 mm/yr (USGS, 1990) is used in the current study, along with a maximum magnitude earthquake of M = 7.5.

Calaveras Fault

A characteristic magnitude of M = 7 and a slip rate of 7.5 mm/yr are assigned to the Calaveras, based on its assumed similarity of behavior to the Hayward Fault (Ake, et al., 1991). Earth Science Associates (ESA) (1982) is cited as a reference. Most recently, two approximately M = 6 events (1979 and 1984) have occurred on its southern portion. For this study, a maximum magnitude of M = 7.25 and a slip rate of 7.5 mm/yr are assigned to this fault.

Healdsburg-Rogers Creek Fault

Ake, et al. (1991), consider this fault system an en-echelon continuation of the Hayward and Calaveras Faults to the south. Therefore, behavior similar to these faults is assumed and a maximum magnitude of M = 7 is assigned. The recently revised slip rate of 9 mm/yr (USGS, 1990) is used in the current study.

Maacama Fault

This fault is also regarded as part of the northward continuation of the Hayward Fault Zone (Herd, 1979); however, a higher characteristic magnitude of M = 7.5 is used by Ake, et al. (1991), after Wesnousky (1986). For this study, a maximum magnitude earthquake of M = 7.5 and a slip rate of 7 mm/yr were used.

Green Valley-Cordelia and Concord Faults

Ake, et al. (1991), consider these faults as a system consisting of multiple rupture segments with characteristic magnitudes of M = 6.5. ESA (1982) is cited as a reference. Slip rates of 4 mm/yr were obtained from Wesnousky (1986). For this study, a maximum magnitude earthquake of M = 6.5 and a slip rate of 4 mm/yr were used for each fault segment.

Marsh Creek, Greenville, and Arroyo Mocho Faults

Ake, et al. (1991), regard these faults as the components of the Greenville Fault system and adopted a characteristic magnitude of M = 6.5 and a slip rate of < 1 mm/yr for them, citing ESA (1982) as a reference. Wong (verbal communication, 1991) indicated a slip rate ranging from 0.1 to 1 mm/yr. For this study, a maximum magnitude earthquake of M = 6.5 and a slip rate of 1 mm/yr were used for each fault segment.

Vaca, Kirby Hill, Antioch, and Davis Faults

Ake, et al. (1991), considered these faults together as a more or less continuous, segmented fault system with similar behavior on each segment, and assigned a characteristic magnitude of M = 6. Wesnousky (1986) and Clark, et al. (1984), are cited as references for the Vaca Fault. Ake, et al. (1991), show uncertainties in slip rates for these faults, listing them all as >0.3 mm/yr. Wong (verbal communication, 1991) gives updated estimates of slip rates for each as follows: (1) The Vaca and Kirby Hill Faults are assigned a slip rate of from 0.02 to 0.1 mm/yr; (2) the Antioch Fault's slip rate is estimated at 0.3 mm/yr; and (3) the Davis Fault's slip rate is estimated at 0.1 mm/yr. In this study, the upper slip rate values were used, and a maximum magnitude earthquake of M = 6.0 was used for Vaca, Kirby Hill, and Davis Faults.

Recent studies by Wills and Hart (1992) conclude that there is no evidence of an active, surface fault along the Antioch Fault trace; however, seismicity indicates that a zone of right-lateral faulting does extend beneath the town of Antioch. Wills and Hart stated that this zone is not in the same orientation as the mapped Antioch Fault, and all of the hypocenters are deeper that 15 km.

In this report, we refer to this seismic zone as the "Antioch" Fault and have assigned it a maximum magnitude earthquake of M = 6.5 and a slip rate of 0.3 mm/yr. For lack of better data, we have continued to use the geometry of the Antioch Fault as input to the HAZARD program for this "Antioch" Fault.

Foothills Fault Zone

Ake, et al. (1991), assigned a characteristic magnitude of M = 6 (also used in this report) for this fault system, site of the 1975 M = 5.7 Oroville earthquake. For this study, a maximum magnitude earthquake of M = 6.5 and a slip rate .007 mm/yr are used.

In this study, the trace of the Bear Mountain Fault Zone is used as the Foothills Fault Zone, with endpoints at Lake McClure in Mariposa County and near Bangor in Butte County (about 10 km southeast of Oroville), where fault displacement occurred during the 1975 Oroville earthquake (see Jennings, 1975).

Midland Fault

The Midland Fault was described by Frame (1944) as a major subsurface structure that was discovered in the Sacramento Delta during the development of the Rio Vista gas field between 1936 and 1943.

The Midland Fault, although buried beneath Eocene and younger sediments in the Delta region, runs northward, directly through the center of this study's area of investigation. However, the fault is considered inactive; displacement apparently last occurred on it between early Paleocene and early Oligocene time (Almgren, 1978).

The Midland Fault lies coincident with, and is enveloped by, the approximately 20-km-wide band of the Coast Range-Sierra Nevada Boundary Zone (see below). The Midland Fault is not used as an earthquake source in this study.

Coast Range-Sierra Nevada Boundary Zone, Segments A, B, and C

Much uncertainty has surrounded the behavior and location of the CRSNB in the past. It has very little surface expression and a very sparse record of seismicity. It is regarded as a 600-km-long zone of complex faulting beneath the Coast Range-Great Valley boundary (Wong, et al., 1988). It has been postulated that this entire feature may be capable of behavior similar to that seen on the Coalinga segment in the southern portion of the CRSNB (slippage on a previously unknown blind thrust fault) during the 1983 M = 6.7 Coalinga Earthquake (Unruh and Moores, 1991).

Ake, et al. (1991), simplified their interpretation of this enigmatic seismic source by treating it as a single, continuous, segmented fault along the western Great Valley Margin. They cited Wong and others (1988) as a reference. However, a more detailed approach, after Wong (written communication, 1991), was used in this investigation.

Wong shows the CRSNB as a multi-segmented band approximately 20 km wide, with each segment having its own slip rate and MCE. Segments A, B, and C apply to this study area and are treated as three separate but contiguous segments running northwest to southeast, with a northern endpoint at the northern terminus of the Sweitzer Fault near Rumsey in Yolo County and a southern endpoint near Los Banos in Merced County. To obtain fault geometry and distances from the DWR Delta sites, the centerline of the 20-km-wide band of the CRSNB was used.

The following additional data were used for the CRSNB: an areal "b" value of 0.785 for segments A, B, and C; a maximum magnitude of M = 6.8 and slip rate of 0.5 to 1.0 mm/yr for Segment A; a maximum magnitude of M = 6.5 and slip rate of 0.1 mm/yr for Segment B; and a maximum magnitude of M = 6.5 and slip rate of 0.5 mm/yr for Segment C. These data are also shown in Table 4-3.

Ortigalita Fault

Ake, et al. (1991), adopted a characteristic magnitude of M = 6.5 for this source and listed the slip rate as <1 mm/yr. Wong (verbal communication, 1991) described this fault as complex, segmented, and very active, assigning a slip rate of approximately 0.1 to 1 mm/yr. In the current study, the conservative figure of 1 mm/yr was used along with a maximum magnitude earthquake of M = 6.75.

2.3 LEVEE HISTORY

In the late 1800s, Delta inhabitants began fortifying existing natural levees and draining inundated islands in the Delta for agricultural use. Figure 2-6 shows the Delta channel system in 1869.

Most of the early levees in the Delta were constructed by Chinese laborers (Thompson, 1982) using hand shovels and wheelbarrows (Figure 2-7), and some were built using scrapers pulled by horses (see Figure 2-8). Later, when the farmers realized that levees of sufficient height could not be efficiently built by hand, the sidedraft-clamshell dredge was used (see Figure 2-9). The levees were generally built of non-select, uncompacted materials without engineering design and without good construction methods. The original levees were usually less than five feet high, but settlement of the levees and subsidence of the interior island soils has required the addition of fill to maintain protection against overtopping by waters of the Delta (see Figure 2-10).

The interiors of many islands are now commonly 10 to 15 feet below sea level. Presently, some levees crowns are 25 feet higher than the interior of their respective islands.

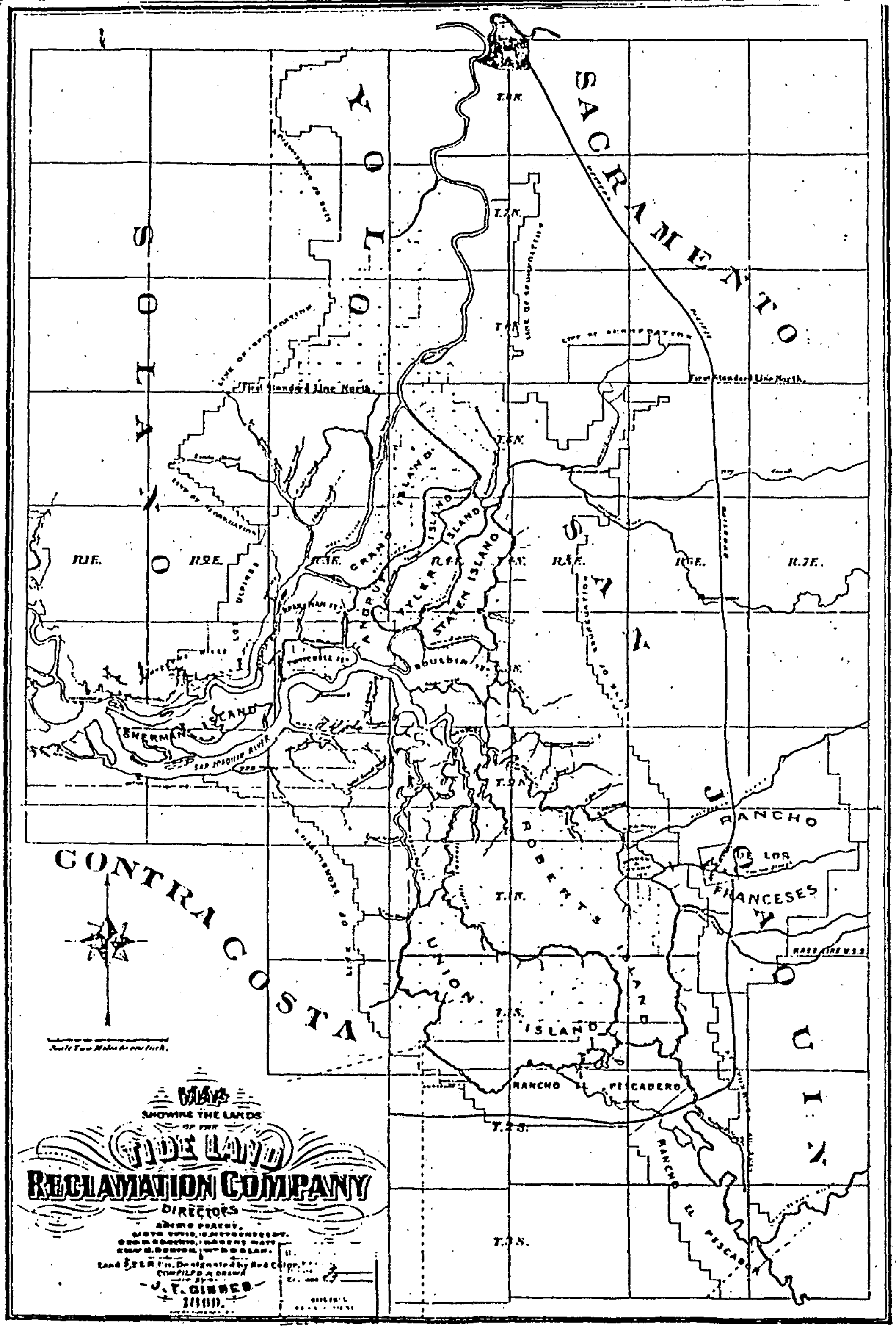


Figure 2-6: The Delta in 1869 (From DWR, 1987)

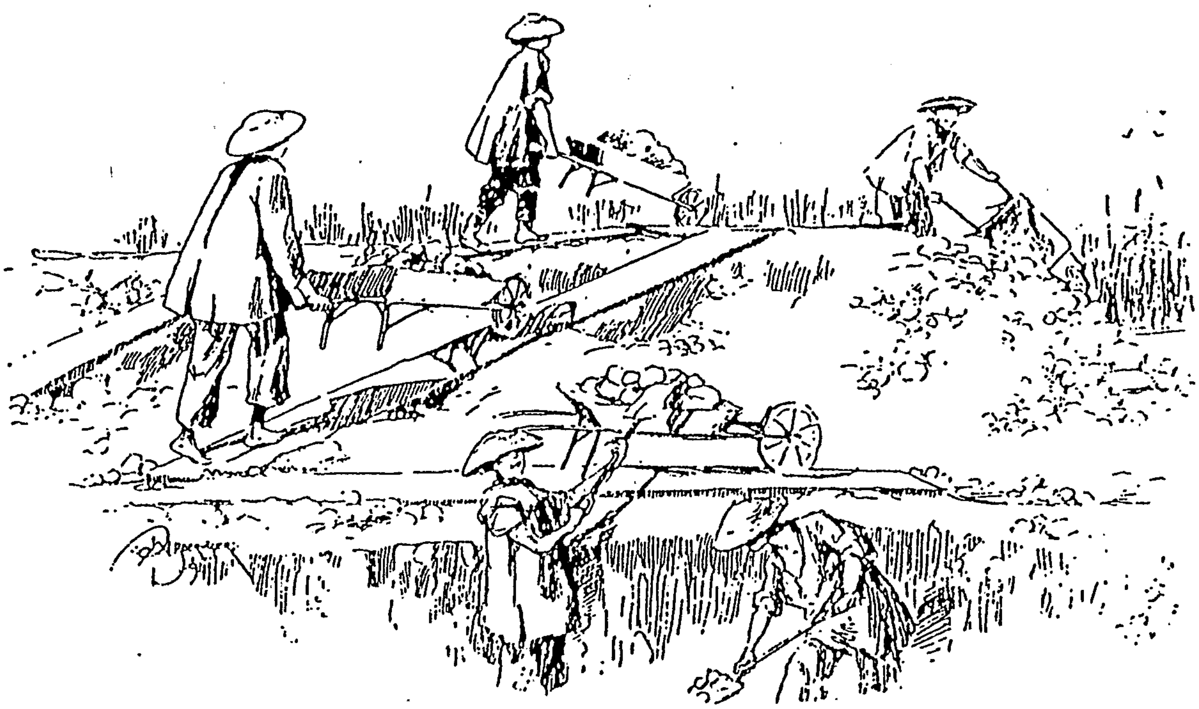
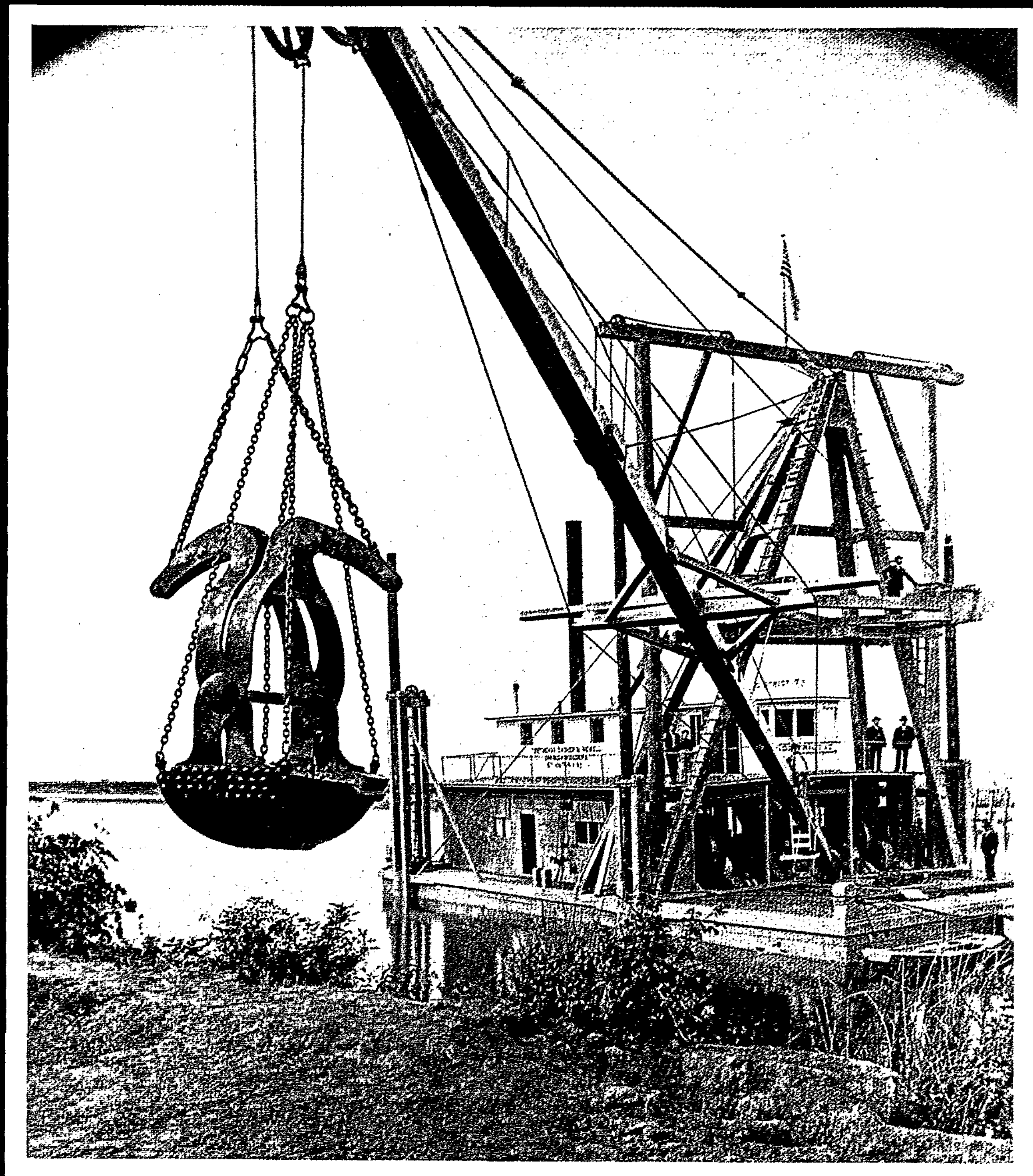


Figure 2-7: Chinese Hand Labor (From DWR, 1982)

Figure 2-8: Horse Drawn Scrapers (From Dutra, 1976)



Dredge DISTRICT NO. 17, later called the BACHMAN, shown near Stockton, California. Built by Tretheway, Dasher & Newell in Stockton in 1893.

Figure 2-9: Sidedraft Clamshell Dredge (From Dutra, 1976)

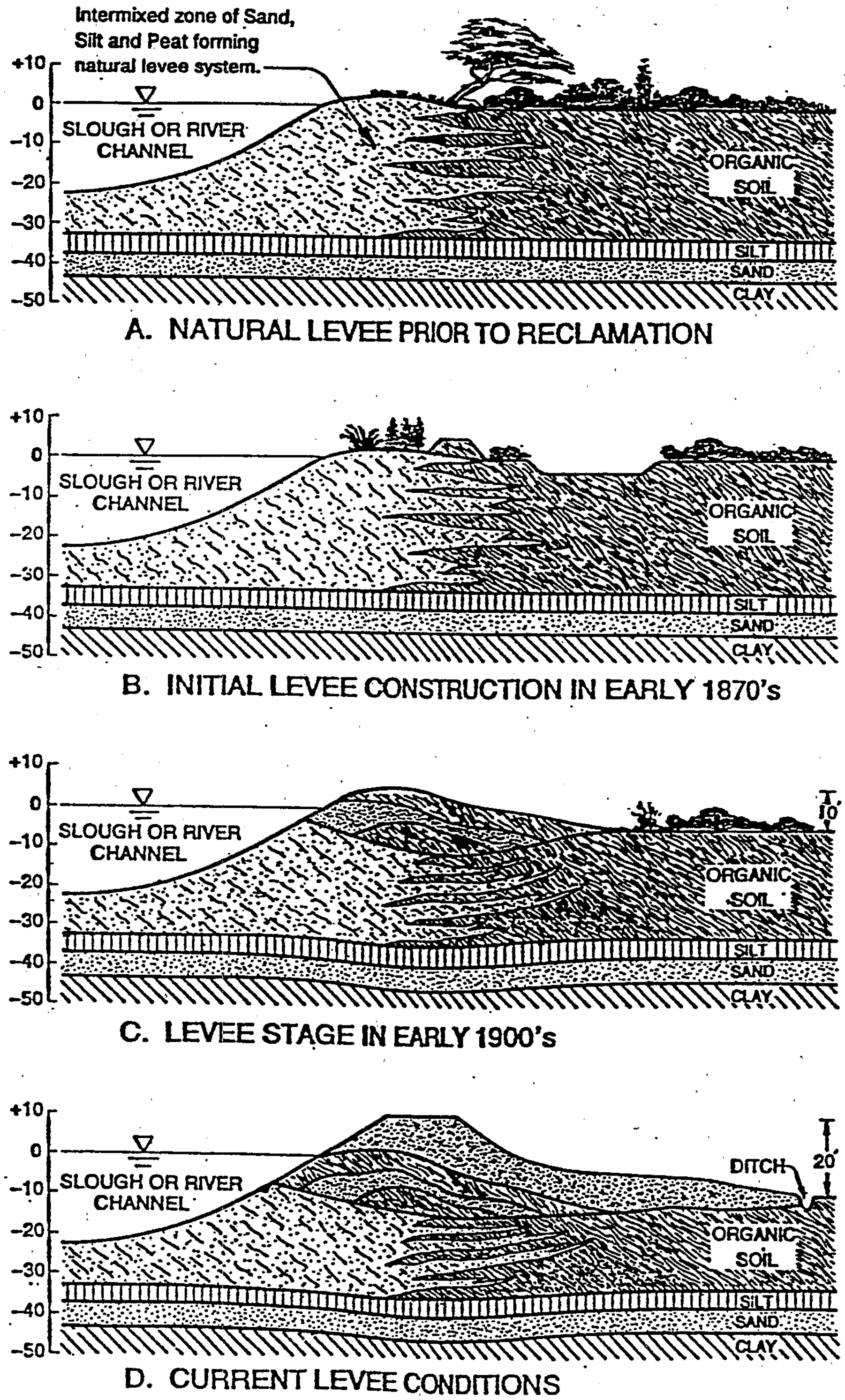


Figure 2-10: Development of Delta Levees, (DWR, 1992)

In general, the upper portions of Delta levee embankments are comprised of mixtures of dredged organic and inorganic sandy, silty, or clayey soils that have been placed on either natural peat or natural sand and silt levees. Figures 2-11 through 2-14 show three examples of levee cross sections and their associated geology. These examples were chosen based on availability and quality of information and to show the variability which exists in the Delta. Figure 2-12 shows a levee and foundation on the south side of Sherman Island, near the Antioch Bridge. This is an area of recent cracking (Foott, 1991). Figure 2-13 shows a levee and foundation near Terminous where State Route 12 crosses Little Potato Slough (Caltrans, 1986). Figure 2-14 shows a levee at Old River at the Mokelumne Aqueduct Crossing (CWDD, 1980). The thickness of organic soils and/or peat at these three sites varies from about 35 feet at Sherman Island to only about 10 feet at Old River. The variability in foundation materials for Delta levees can be great, even between sites that are in close proximity to one another. Such heterogeneity is due to a history of continuous stream meandering and channel migration within the Delta.

The Delta levees are considered susceptible to earthquake induced failures because many incorporate and are founded on substantial zones of loose, saturated, cohesionless soils. Such soils are known to be susceptible to liquefaction during earthquakes (Seed, 1985). Further, many of the levees are partially composed of, and built on, soft organic soils which are highly compressible with low shear strengths. However, the behavior of organic soils and fibrous peats during earthquakes is not well understood.

Cracks on levees for only steady state static loading are very common. They can be caused by differential settlement, and can be triggered by changes in loading conditions. Since the first levees were constructed in the mid-1800s there have been numerous non-earthquake related failures. Generally, these failures are a result of overtopping or instability. Table 2-2 presents a partial listing of past Delta levee failures.

Figure 2-16 depicts the frequency of island inundations using the same historical data presented in Table 2-2. This figure indicates that the high rainfall experienced in the early to mid-1980s was responsible for an accelerated rate of levee failure. Due to continued subsidence, even higher rates of failure can be expected during such periods of high water in the future unless the levee systems are substantially upgraded. Although levee failures in the past were often caused by floodwaters overtopping levee crowns, the more common mode of levee failure in recent years is considered to be stability failures, with piping being partially responsible (Duncan, et al., 1980).

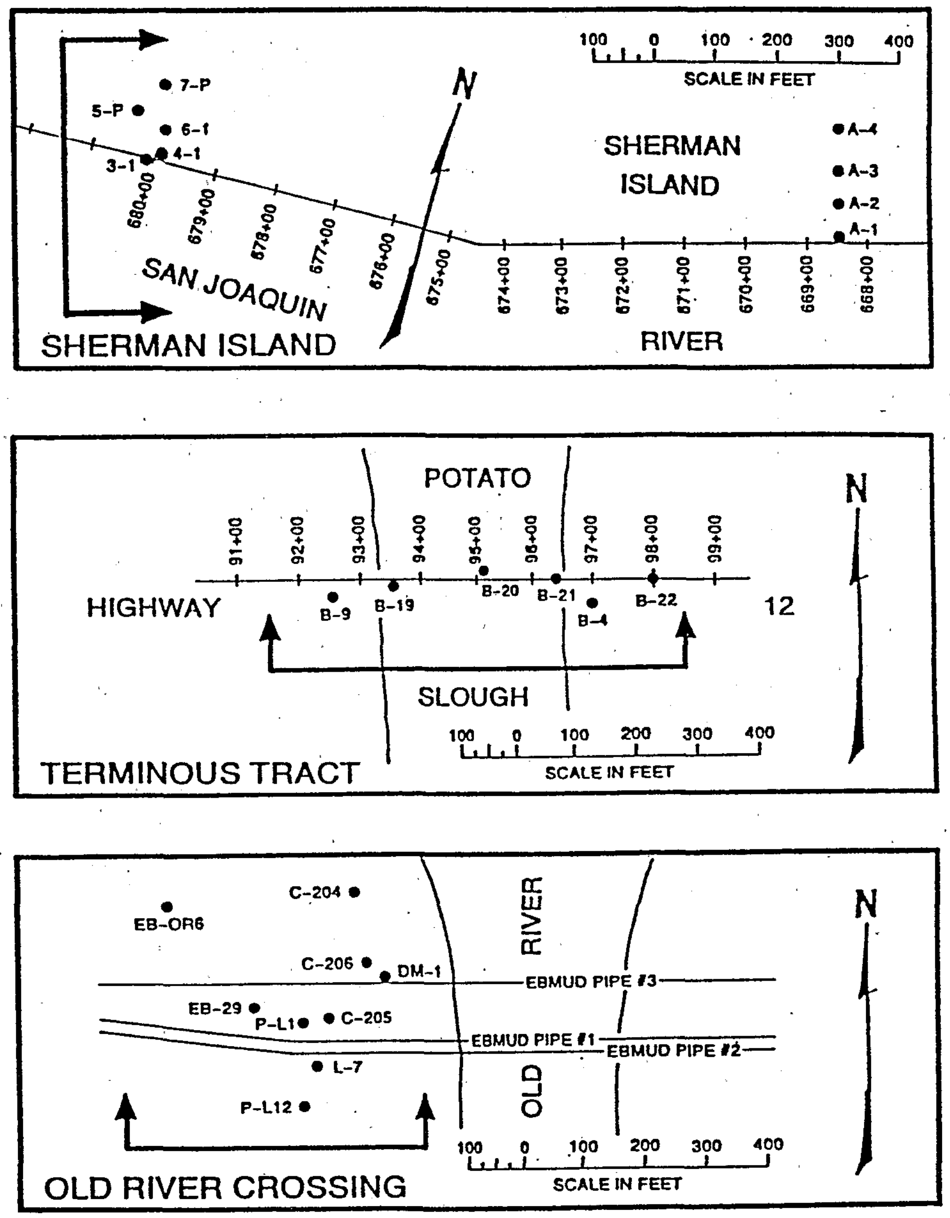
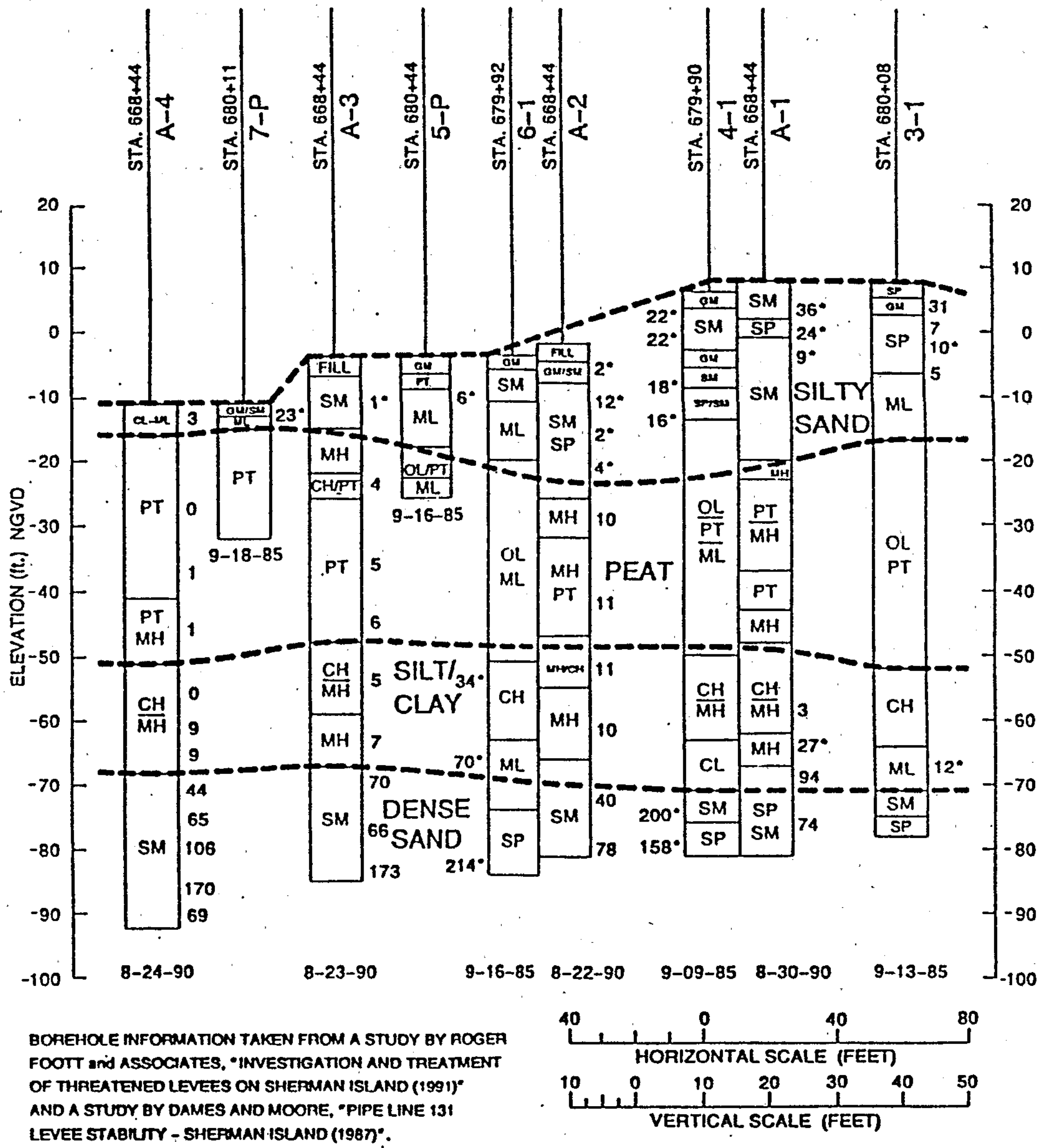


Figure 2-11: Locations of Boreholes at Selected Delta Sites

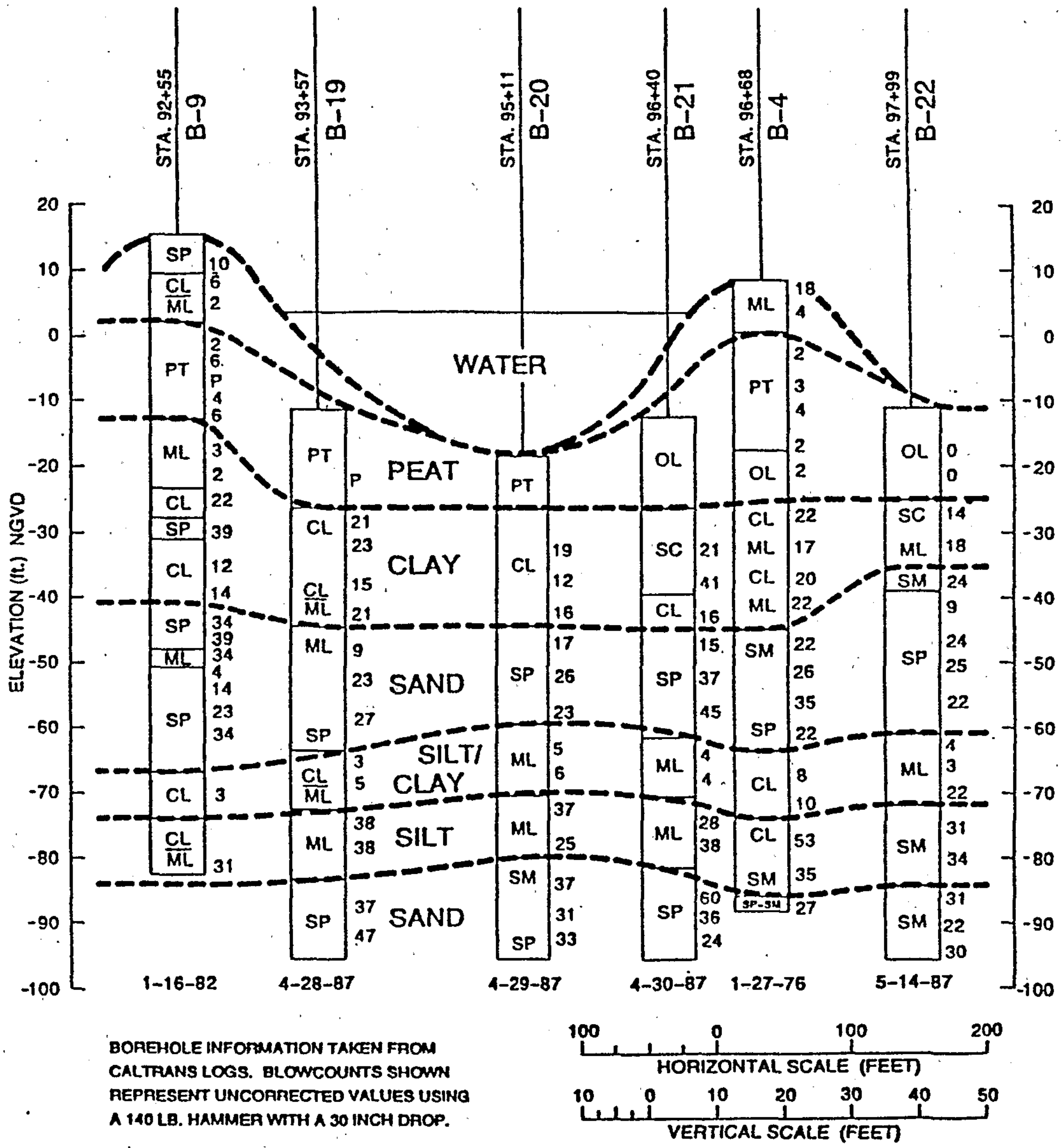


BOREHOLE INFORMATION TAKEN FROM A STUDY BY ROGER FOOTT and ASSOCIATES, "INVESTIGATION AND TREATMENT OF THREATENED LEVEES ON SHERMAN ISLAND (1991)" AND A STUDY BY DAMES AND MOORE, "PIPE LINE 131 LEVEE STABILITY - SHERMAN ISLAND (1987)".

BLOWCOUNTS SHOWN REPRESENT UNCORRECTED VALUES USING A 140 LB. HAMMER WITH A 30 INCH DROP. ASTERISK (*) INDICATES NON-STANDARD SPT.

NOTES:
1. HORIZONTAL SCALE IS APPROXIMATE.
2. REFER TO FIGURE 2-13 FOR PLAN VIEW OF SECTION.

Figure 2-12: Levee Cross-Section at Sherman Island



BOREHOLE INFORMATION TAKEN FROM CALTRANS LOGS. BLOWCOUNTS SHOWN REPRESENT UNCORRECTED VALUES USING A 140 LB. HAMMER WITH A 30 INCH DROP.

NOTES:
1. HORIZONTAL SCALE IS APPROXIMATE.
2. REFER TO FIGURE 2-13 FOR PLAN VIEW OF SECTION.

Figure 2-13: Levee Cross-Section near Terminous

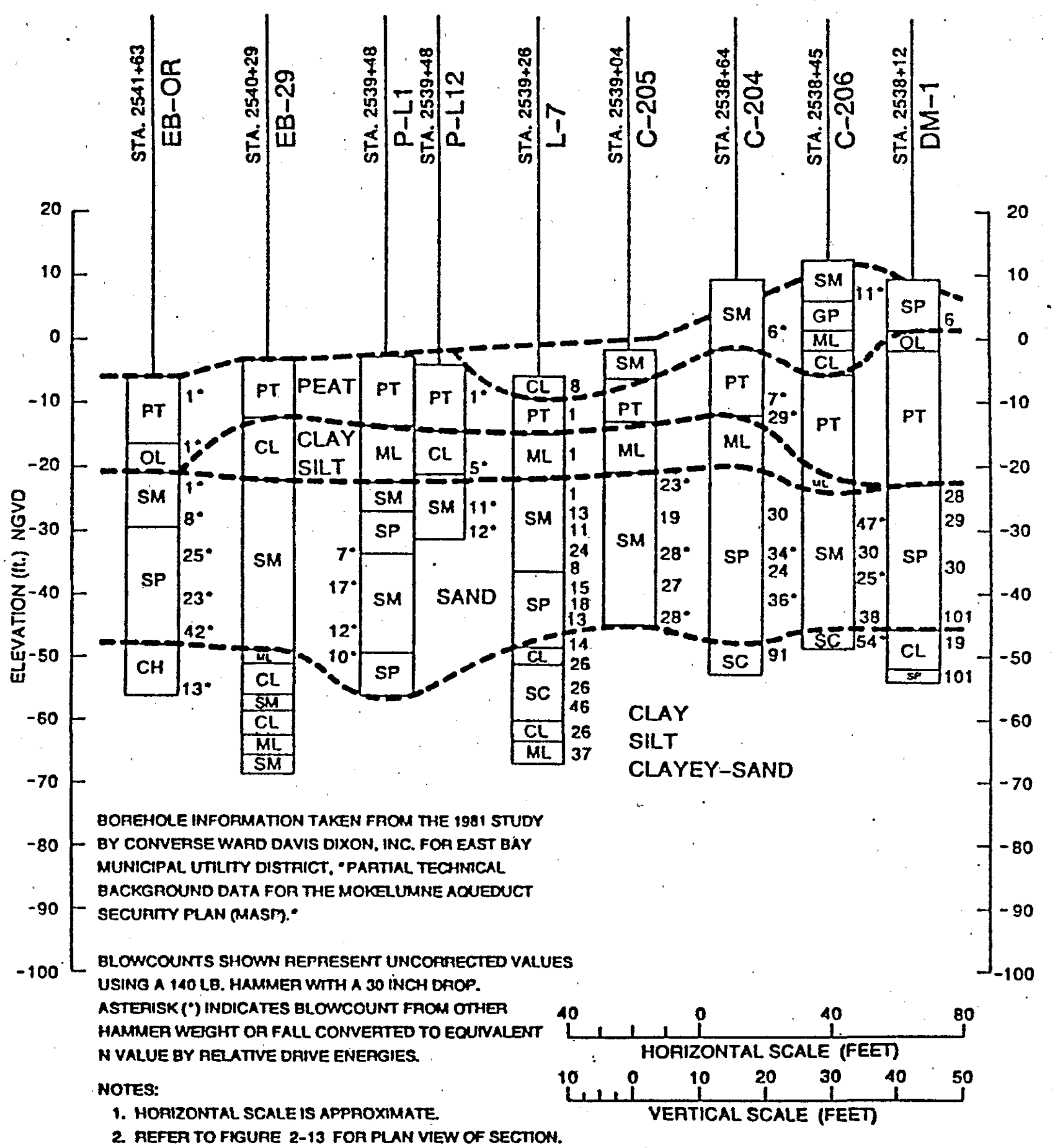
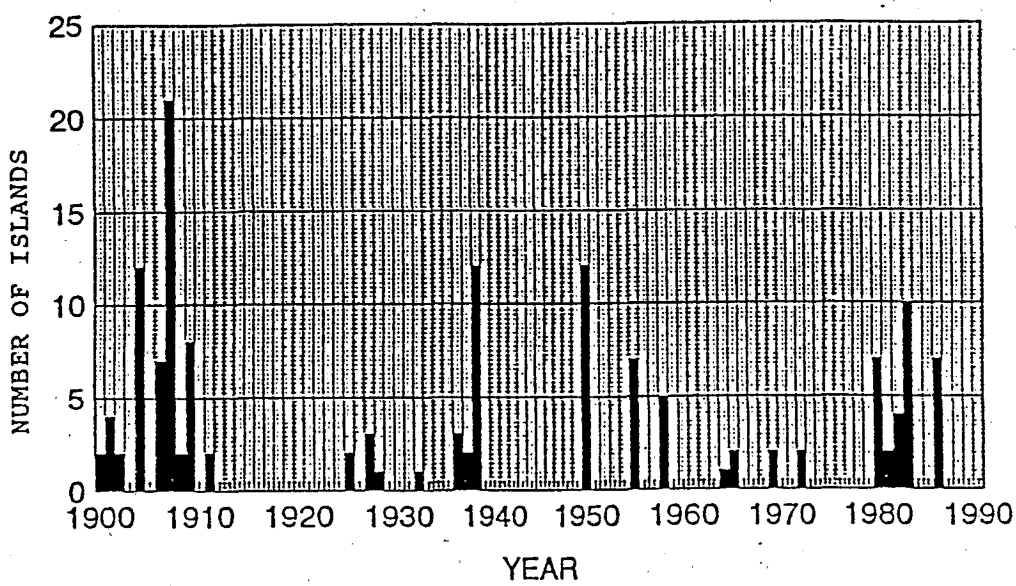


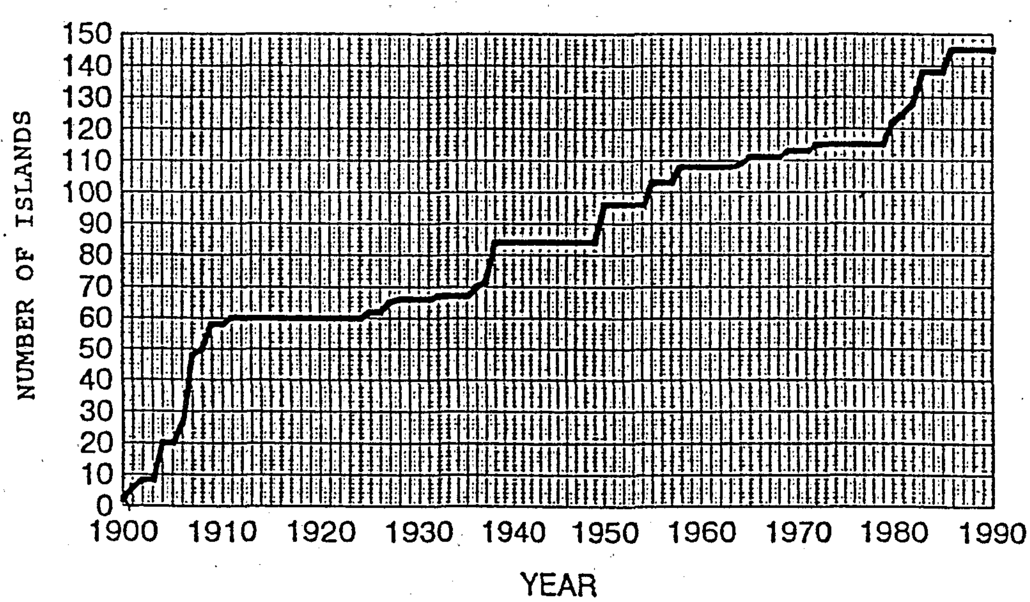
Figure 2-14: Levee Cross-Section at Old River and the Mokelumne Aqueduct Crossing

	AREA FLOODED	ACRES FLOODED	YEAR FLOODED
9	ANDRUS ISLAND	7,200	1902-1907-1909-1972
	BACON ISLAND	5,546	1938
	BETHEL ISLAND	3,400	1907-1908-1909-1911-1972
2	BIG BREAK	2,200	1927
	BISHOP TRACT	2,100	1904
8	BOULDIN ISLAND	5,600	1904-1907-1908-1909-1972
	BRACK TRACT	2,500	1904
4	BRADFORD ISLAND	2,000	1950-1983
11	BRANNAN ISLAND	7,500	1902-1904-1907-1909-1972
	BYRON TRACT	6,100	1907
	CANAL RANCH TRACT	500	1958-1986
	CLIFTON COURT TRACT	3,100	1901-1907
	CONEY ISLAND	900	1907
	DEAD HORSE ISLAND	200	1950-1955-1958-1980-1986
	DONLON ISLAND	3,000	1937
	EDGERLY ISLAND	150	1983
	EMPIRE TRACT	3,500	1950-1955
	FABIAN TRACT	6,200	1901-1906
	FAY ISLAND	100	1983
5	FRANKS TRACT	3,300	1907-1936-1938
	GRAND ISLAND	-	1955
	GRIZZLY ISLAND	8,000	1983
	HOLLAND TRACT	1,100	1980
	IDA ISLAND	100	1950-1955
3	JERSEY ISLAND	3,400	1900-1904-1907-1909
	LITTLE FRANKS TRACT	350	1981-1982-1983
	LITTLE MANDEVILLE ISLAND	200	1980–
	LOWER JONES TRACT	5,700	1907-1980
	LOWER ROBERTS ISLAND	10,300	1906
	LOWER SHERMAN ISLAND	3,200	1907-1925
	MANDEVILLE ISLAND	5,000	1938
	MC CORMACK-WILLIAMSON TRACT	1,500	1938-1950-1955-1958 1964-1986
	MC DONALD ISLAND	5,800	1982
	MEDFORD ISLAND	1,100	1936-1983
	MIDDLE ROBERTS ISLAND	500	1938
	MILDRED ISLAND	900	1965-1969-1983
	NEW HOPE TRACT	2,000-9,500	1900-1904-1907-1928-1950 1955-1986
	PALM TRACT	2,300	1907
	PESCADERO	3,000	1938-1950
	PROSPECT ISLAND	1,100	1980-1981-1982-1983-1986
	QUIMBY ISLAND	700	1936-1938-1950-1955
	RD 17	4,500-5,800	1901-1911-1950
	RD 1007	3,000	1925
	RHODE ISLAND	100	1938
	RYER ISLAND	11,600	1904-1907
	SARGENT BARNHART TRACT	1,100	1904-1907
1	SHERMAN ISLAND	10,000	1904-1906-1909-1937-1969
	SHIMA	2,394	1983
	SHIN KEE TRACT	700	1938-1958-1965-1986
	STATEN ISLAND	8,700	1904-1907
	STEWART TRACT	3,900	1938-1950
	TERMINOUS TRACT	5,000-10,000	1907-1958
10	TWITCHELL ISLAND	3,400	1906-1907-1909
	TYLER ISLAND	8,700	1904-1907-1986
	UNION ISLAND	24,000	1906
	UPPER JONES	6,200-5,700	1906-1980
	UPPER ROBERTS	500	1938
	VAN SICKLE	-	1983
7	VENICE ISLAND	3000	1904-1906-1907-1909-1932 1938-1950-1982
	VICTORIA ISLAND	7,000	1901-1907
6	WEBB TRACT	5,200	1950-1980

Table 2-2: Islands Flooded since 1900



A) NUMBER OF DELTA ISLANDS FLOODED EACH YEAR SINCE 1900



B) ACCUMULATED NUMBER OF DELTA ISLAND INUNDATIONS SINCE 1900

Figure 2-15: Historical Summary of Flooded Delta Islands

There have been no known earthquake-induced levee failures in the Delta. However, this kind of failure is common to earth embankments built on soft, marshy soil. Since 1906, when the levees were considerably smaller, there has not been a large earthquake near the Delta. This is discussed further in Chapter 5.

3. REVIEW OF REPORTS

3.0 INTRODUCTION

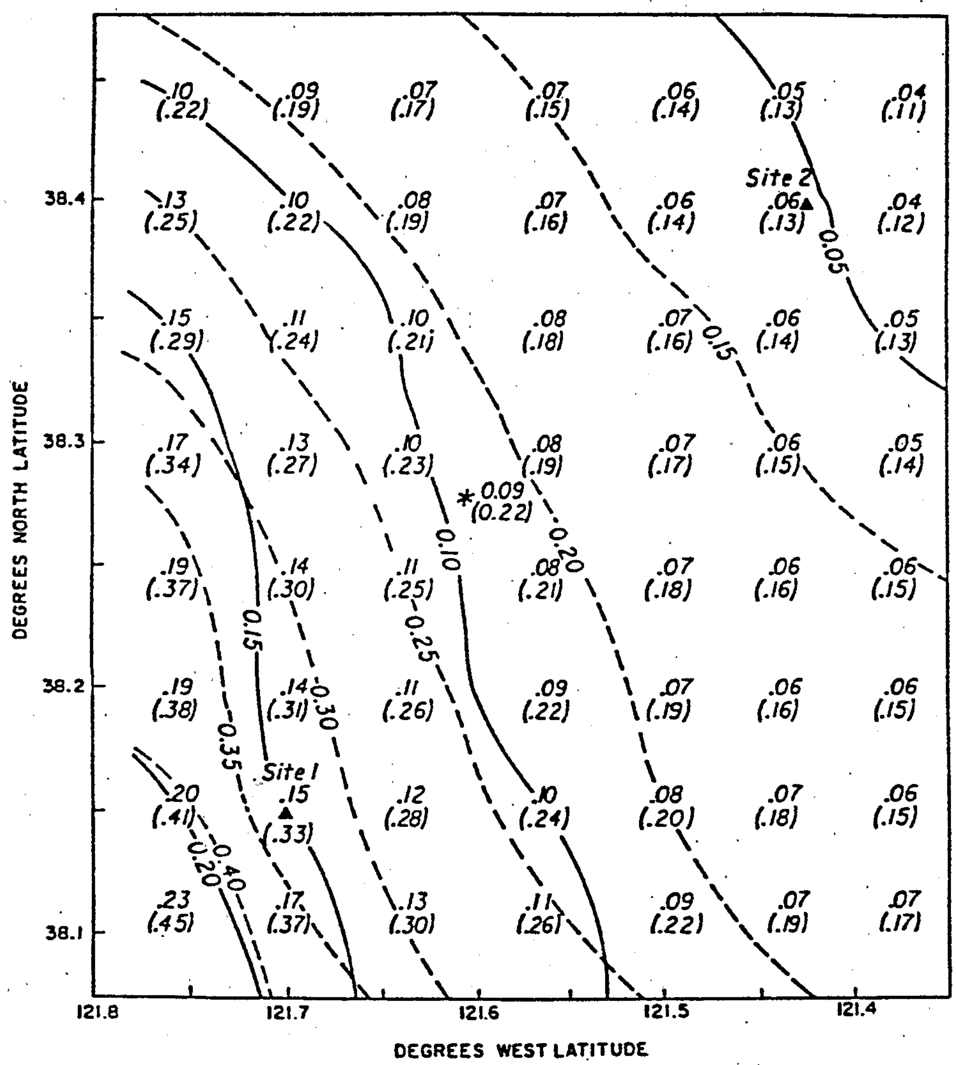
Several reports concerning seismic hazards and risk analysis have been prepared for the Delta region during the last 12 years by government and private concerns. The Department of Water Resources has prepared two reports, "Delta Seismicity" in 1980 and an unpublished report, "Seismicity" in 1985. The Department has also co-funded a preliminary seismic risk analysis performed by the U. S. Bureau of Reclamation (Ostenaa, et al., 1989) as part of the EIR for the South Delta Water Management Study, and a similar study for the North Delta Water Management Study (Ake, et al., 1991).

All of these previous studies are preliminary in nature due to the lack of reliable data for the vast Delta levee system. A comprehensive review of many of these reports was performed for this study examining the types of evaluations made and the data that was developed. Table 3-1, presented at the end of this chapter, summarizes this review. Listed below are summary descriptions of those previous studies.

3.1 PRELIMINARY SEISMIC RISK ANALYSIS FOR THE DELTA WATER MANAGEMENT STUDY: NORTH DELTA
U. S. Bureau of Reclamation; Department of Water Resources, September 1991

This report was written by Ake, et al. (1991). and presents a preliminary seismic risk assessment of levees in the North Delta. These studies were co-funded by the Bureau of Reclamation and the Department of Water Resources for use as input to the environmental impact statement/environmental impact report as part of the Delta Water Management Study.

The principal focus of the studies was to develop peak horizontal ground accelerations for an exposure period of 100 years, with a probability of non-exceedance of 90 percent. These determinations were made using computer program SEISRISK III with the attenuation curves developed by Joyner and Boore (1981). No allowance was apparently made for either significant amplification or damping through the soft, organic soils present beneath the levee system. The results indicate that mean peak horizontal ground accelerations would range from about 0.05g to 0.25g, generally decreasing in acceleration from southwest to northeast (see Figure 3-1).



Peak acceleration (in g's) with 90 percent probability of non-exceedance in 100-year exposure period, North Delta study area. Attenuation relationships of Joyner and Boore (1981) used, mean values with +1 standard deviation values shown in parentheses. Contours for mean values shown as solid lines, contours for +1 standard deviation values as dashed lines.

Figure 3-1: Peak Accelerations, (from USBR, 1991)

Preliminary geotechnical analyses were also performed, including liquefaction and deformation analyses. Although many of the details of these analyses were not presented, the studies apparently made use of previous liquefaction determinations and data developed in the 1987 U. S. Army Corps of Engineers liquefaction study (see Section 3.8). The 1991 Bureau study very conservatively assumed that levee failure would occur at half of the levee sites where potentially liquefiable soils are present. In addition, it was assumed that a calculated deformation of one foot would result in levee failure. These analyses led to predictions of levee failure for four zones across the North Delta region (see Figure 3-2). Percentages of levees predicted to fail for the 100-year exposure period corresponded to different levees of ground motion:

Horizontal Peak Acceleration (g)	Percentage of Delta Levees Expected to Undergo Liquefaction-Induced Failure (%)
0.04 - 0.05	0 - 8
0.05 - 0.10	8 - 23
0.10 - 0.20	23 - 31
0.20 - 0.23	31 - 36

The horizontal accelerations presented in this report are generally in the range estimated by similar reports for either still soil or bedrock. The general prediction of liquefaction-induced failure is also in line with most other studies. However, the study indicates that liquefaction-induced failure would occur for acceleration values as low as .05g. This would appear to be in conflict with the past good performance of the levees for similar levels of motion. This result, along with the other assumptions regarding liquefaction and deformation failure criteria, appears to be very conservative. On the other hand, the assumption of no ground motion amplification through the organic soils may be unconservative.

The report lists a variety of limitations regarding the studies performed and provide recommendation for future studies. Many of these recommendations are similar to those being recommended for Phase II of the current investigation (see Chapter 8).

3.2 CALIFORNIA'S WATER FUTURE, AN OVERVIEW AND CALL TO ACTION Association of California Water Agencies, July 1991

This publication is a booklet that was part of an information packet presented to the California Seismic Safety Commission during a September 1991 commission meeting. The booklet discusses a

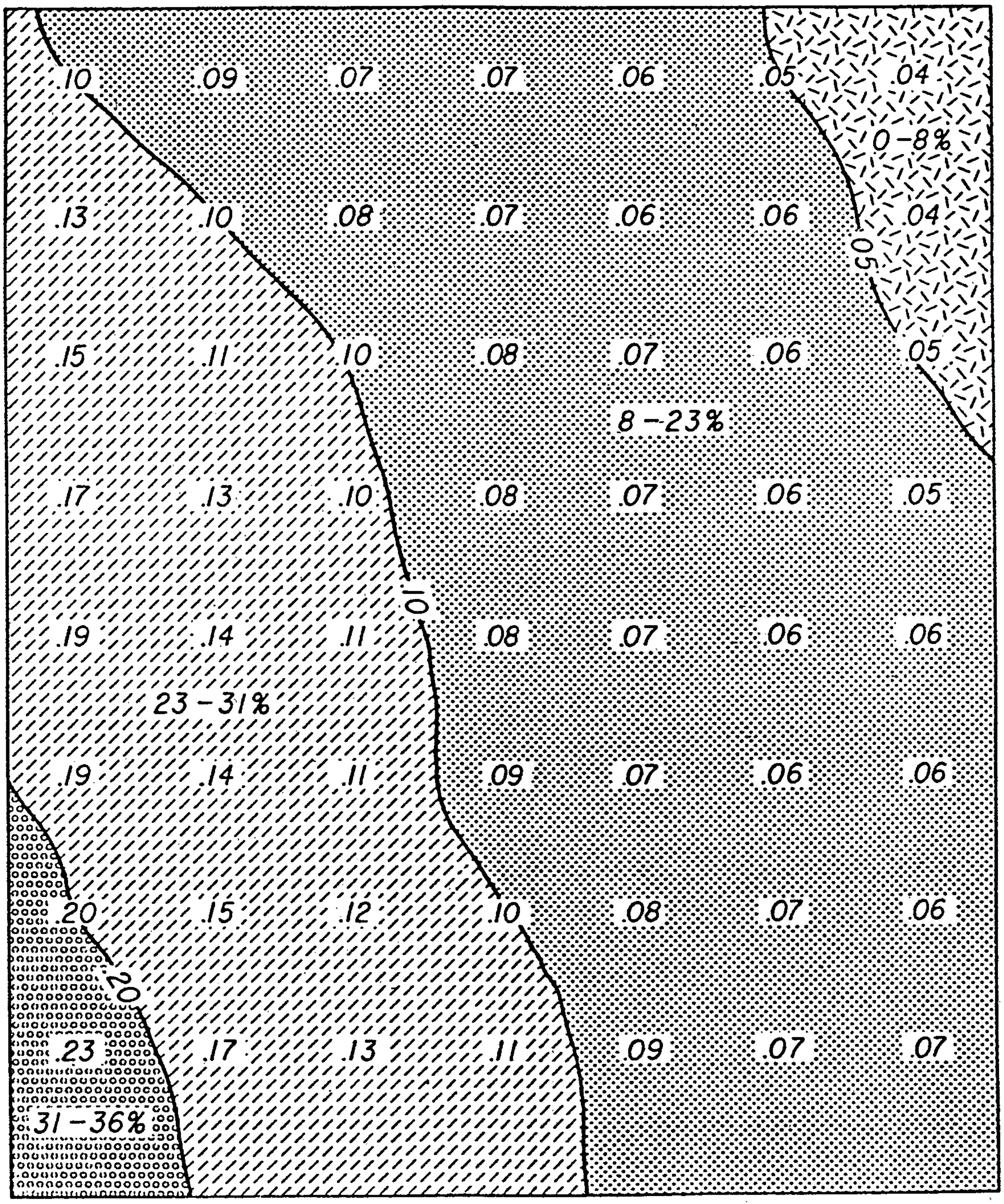


Figure 3-2: Liquefaction Potential in the North Delta. Percentage of Delta Levees Expected to Undergo Liquefaction-Induced Failure, Based on Peak Acceleration Values.

C-072275

variety of issues that impact California's water supply and water distribution system. It briefly addresses the earthquake risk by stating that "there is certainly a risk that such an earthquake could destroy the Delta levee system...," referring to an earthquake that is a Magnitude 7.0 event "in or near the Delta" occurring within the next thirty years.

The information packet also contained copies of figures from an unpublished seismic risk study of the Mokelumne Aqueduct by Earth Sciences and Associates, together with figures showing an inundated Delta previously published by Miller (1990).

The scenario created by the booklet and accompanying figures indicates that much or most of the Delta islands would be flooded as a result of near-future earthquake shaking. It is difficult to evaluate the predictions made, as there was no report or backup data available to examine. It appears from some of the figures that significant amplification is being assumed for the organic soils beneath the levees, and that the Coast Range-Sierra Nevada Boundary Zone is considered to be a major earthquake source.

3.3 GENERAL SEISMIC AND GEOTECHNICAL RISK ASSESSMENT, SACRAMENTO-SAN JOAQUIN DELTA, CALIFORNIA
Dames and Moore, March 6, 1991

The purpose of this study was to provide an overview of seismic and geotechnical risks for facilities located in the Delta. This report was performed for the State Water Contractors Association and provides a brief review of historic earthquake damage in the Delta, develops a range of bedrock accelerations obtained by using statistical procedures based on methods by Cornell and Vanmarcke (1969), and expanded by Donovan and Bornstein (1975), and discusses general geotechnical problems encountered in the Delta. Much of the report appears to incorporate work from earlier study by Dames and Moore for McDonald Island.

The report of historic earthquake damage in the Delta is based primarily on the 1980 seismicity hazard study jointly conducted by the California Department of Water Resources and the U. S. Army Corps of Engineers, and the February 1985 issue of California Geology which contains the report "Earthquake Related Damage, Sacramento-San Joaquin Delta," by Michael Finch.

The generalized seismic hazard assessment utilizes probabilistic methods to calculate accelerations values for the 100-year exposure period at a ten percent probability of exceed, and for a 50-year exposure period at a 50 percent probability of exceedance. These are presented as contours across the Delta region (see Figures 3-3 and 3-4). The section on geotechnical problems encountered in the Delta discusses problems associated with Delta levees; however, most of this discussion is not related to earthquakes.

3.4 A NEW VIEW OF THE SACRAMENTO-SAN JOAQUIN DELTA
Dr. B. J. Miller, Consulting Engineer
May 8, 1990

This paper presents a very general discussion of problems facing the Delta waterways, including problems associated with fisheries, drinking water, and earthquakes. The earthquake discussion consists of general statements describing two scenarios of how the Delta would look following the onset of earthquakes that would be expected to occur within the next 30 years. The postulated scenarios for the Delta both essentially result in most of the islands being inundated (see Figures 3-5 and 3-6). The supporting analyses and evaluations are not included. It is, however, unlikely that any one earthquake event could cause such a result.

As for the 1991 ACWA information packet, it is difficult to evaluate the predictions being made as the supporting information is not available to examine. However, Dr. Miller does make several statements regarding the availability of good geotechnical data across the Delta and current knowledge with respect to predicting the onset of liquefaction in the Delta. His statements allude to a high confidence in both the data and in the understanding of ground motion amplification in Delta soil profiles. Most other studies conclude that the available data are limited in both quantity and quality, and that there is considerable uncertainty regarding the potential amplification/attenuation characteristics of soil profiles in the Delta.

3.5 SEISMIC DESIGN CRITERIA, WILKERSON DAM, BOULDIN ISLAND, CALIFORNIA - DRAFT
Harding Lawson Associates
April 3, 1990

The purpose of this report was to present the seismic design criteria and the analytical design procedure developed for evaluating the seismic stability of the proposed Wilkerson Dam on Bouldin Island, California. This report was prepared for the Delta Wetlands Project for submittal to the Department of Water Resources, Division of Safety of Dams for their review and concurrence. The report proposes an approach for evaluating the performance of the foundation soils during the design earthquake

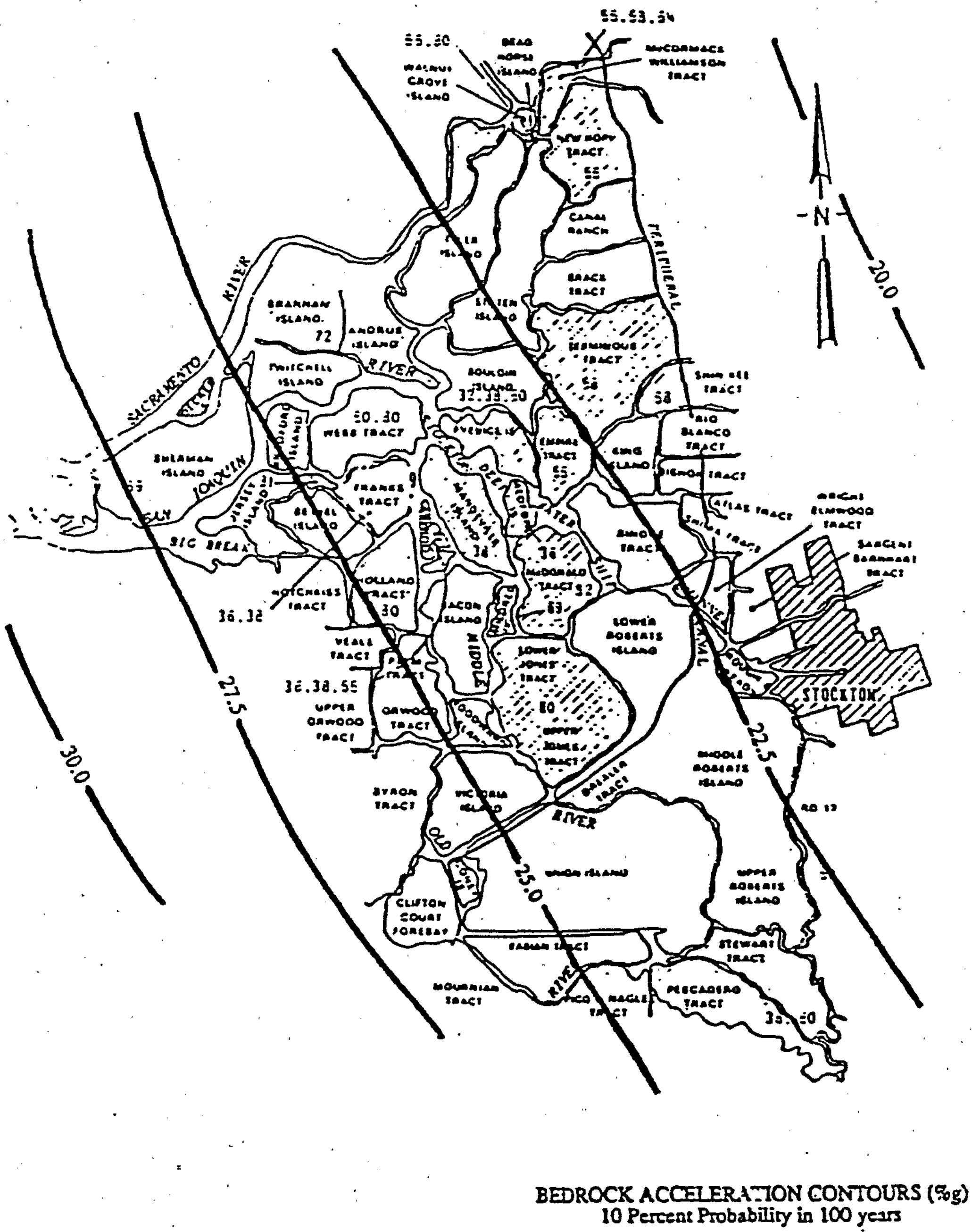


Figure 3-3: Bedrock Acceleration Contours, (from Dames and Moore, 1991).

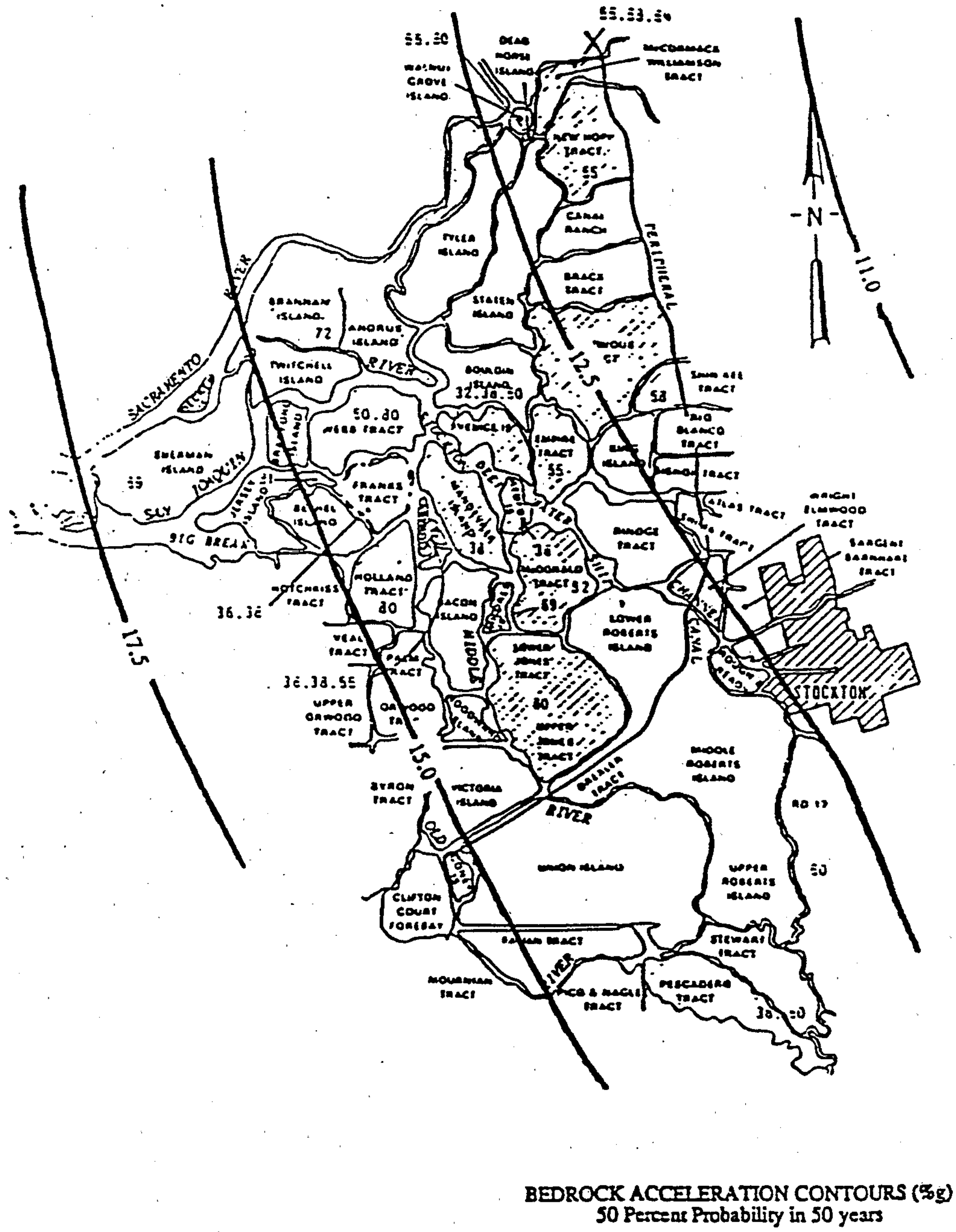


Figure 3-4: Bedrock Acceleration Contours, (from Dames and Moore, 1991).

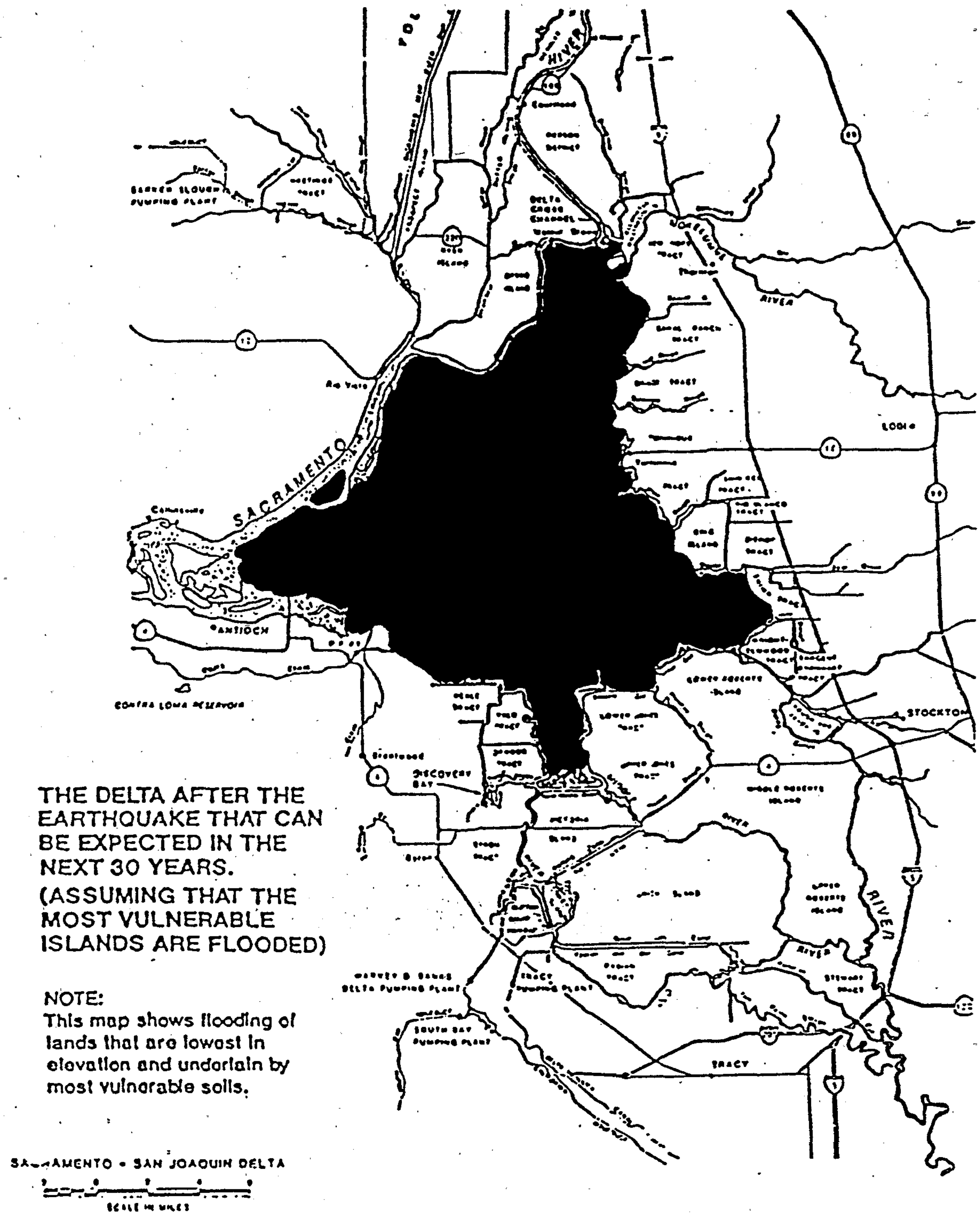


Figure 3-5: The Delta After the Earthquake, (from B.J. Miller, 1990).

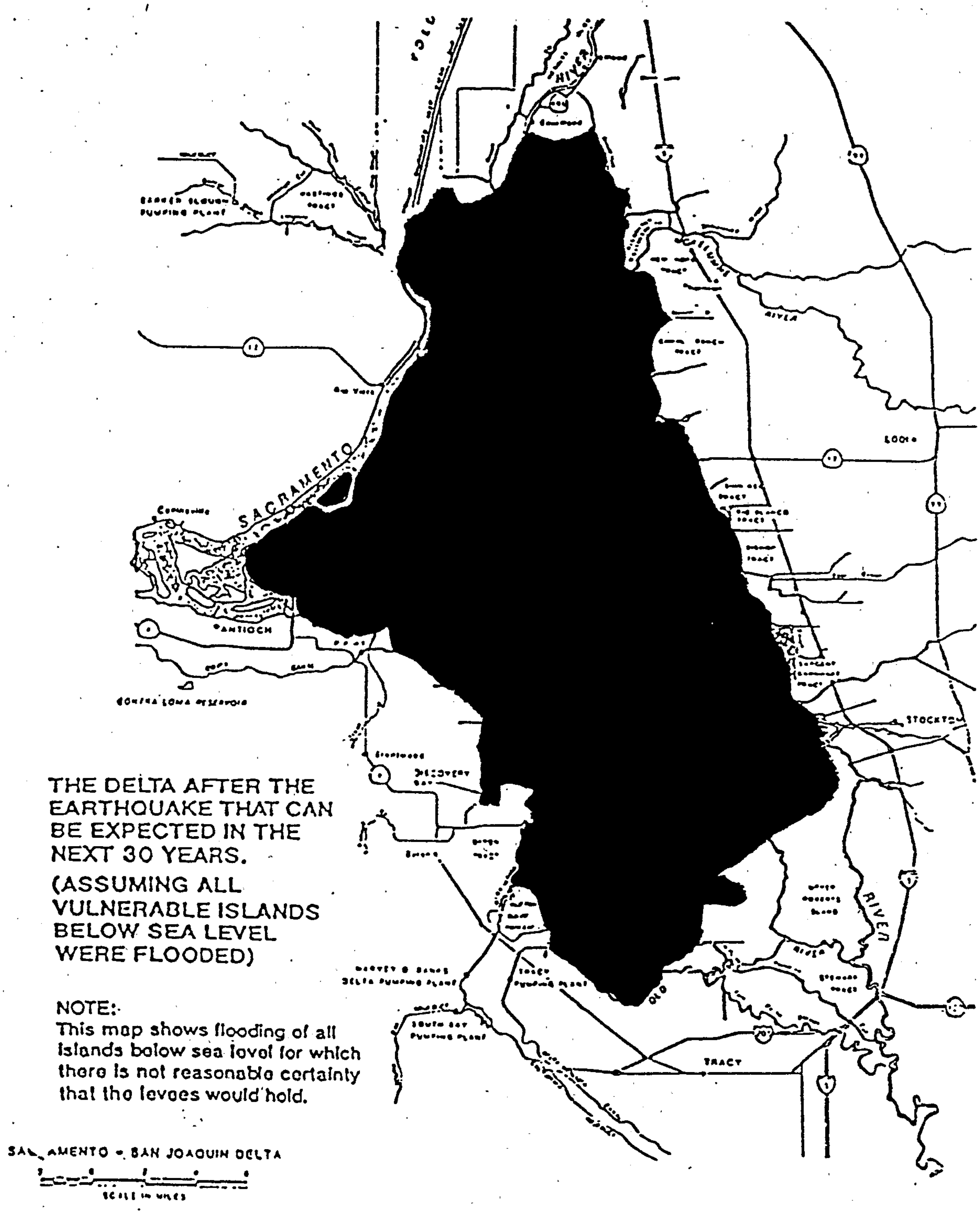


Figure 3-6: The Delta After the Earthquake, (from B.J. Miller, 1990).

by: 1) field investigations, 2) in-place and laboratory soil testing, 3) geological and seismological studies, and 4) site response and dynamic slope stability analyses. The report also presents seismic design criteria and the design ground motion.

The proposed methods for evaluating the performance of the dam under seismic loading are specific and propose to include evaluation of peak bedrock acceleration corresponding to the maximum credible earthquake. The acceleration time histories selected include two from the 1989 Loma Prieta Earthquake (the rock motion recorded at Santa Cruz, and the rock motion recorded at Yerba Buena Island) and two motions developed by Seed, et al., to represent bedrock motion caused by maximum credible events on the San Andreas and Hayward faults. Other proposed tasks include the determination of dynamic properties of the different soil types at the site, one dimensional dynamic response analyses, evaluation of liquefaction potential, and a deformation analysis using the method developed by Makdisi and Seed.

Peak bedrock accelerations were developed by deterministic and probabilistic methods. The probabilistic method gave a higher value, 0.25g with a return period of 500 years vs. 0.21g for a Magnitude 7 earthquake on the Antioch Fault. Attenuation relationships developed by Joyner and Boore (1981) were used.

3.6 PRELIMINARY SEISMIC RISK ANALYSIS FOR THE DELTA WATER MANAGEMENT STUDY: SOUTH DELTA U. S. Bureau of Reclamation December 1989

This report was written by Ostenaa, et al. (1989), and presents a preliminary seismic risk assessment of typical water management facilities within the South Delta study area. It is the companion report to the North Delta report mentioned in Section 3.1. This report was also used as input to the environmental impact statement/environmental impact report prepared as part of the Delta Water Management Study: South Delta. The report determines peak ground accelerations by probabilistic methods using attenuation relationships by Joyner and Boore (1981) and Seed and Idriss (1982).

As for the North Delta investigation, the primary focus of this study was to estimate peak horizontal ground accelerations. This study also did not appear to consider the potential for the organic soils in the Delta to either significantly amplify or attentuate earthquake motions. The peak horizontal ground accelerations for an exposure period of 100 years and a probability of non-exceedance of 90 percent was 0.18g.

Preliminary liquefaction and deformation analyses were also performed for this study. Very little geotechnical data were available to support the parameters used in these calculations. A calculated deformation of one foot was again used to define deformation failure. The study concludes that the percentage of levees in the South Delta area that would fail from future earthquakes within a 100-year exposure period would be as follows:

Earthquake-Induced Failure Mechanism	Percentage of Delta Levees That Would Undergo Failure (%)
Liquefaction of Levee Foundation	20
Liquefaction of Levee Embankment	5 - 10
Deformation Failure	10

3.7 ESTIMATED PERFORMANCE OF TWITCHELL ISLAND LEVEE SYSTEM, SACRAMENTO-SAN JOAQUIN DELTA UNDER MAXIMUM CREDIBLE EARTHQUAKE CONDITIONS.
Bulletin of the Association of Engineering Geologists
Vol. XXV, No. 2, 1988

This paper presents a seismic stability analysis of a berm on Twitchell Island's Three Mile Slough Levee. The "Simplified Procedure for Evaluating Liquefaction Potential" by Seed, et al. (1985), was used to calculate liquefaction potential and deformations were estimated by methods developed by Kutter (in development at the time of the report).

Peak accelerations were estimated using the Maximum Credible Earthquake and the Joyner and Boore (1981) attenuation relationship, and range from 0.26g for the San Andreas Fault to 0.5g for the Midland Fault. The report concludes that liquefaction of a sandy area occurs based on these analyses. The deformation analysis presented estimates displacements of 14,000 (yes, fourteen thousand) feet. Although the displacements were not credible, the report concludes that the stabilizing berm displaces enough to result in failure of the levee.

3.8 SACRAMENTO-SAN JOAQUIN DELTA LEVEE LIQUEFACTION POTENTIAL
Corps of Engineers, Sacramento, California
April 1987

This report utilizes available borehole exploration data to predict the liquefaction potential of the Delta levees and foundations. Most of the available penetration test data were relatively old and obtained with non-standard equipment. Consequently, numerous corrections were required. In many cases, the actual test procedures and corrections had to be estimated. The method of analysis used in this study is the procedure outlined by Seed, et al. (1985). Three maximum accelerations values were assumed (0.05g, 0.10g, and 0.20g).

This report concludes that based on available boring data and the results of the liquefaction analyses, many of the Delta island levees are constructed of or founded on potentially liquefiable deposits of sand and silt (see Figure 3-7). Seven islands were considered to have high liquefaction potential, as evidenced by calculations showing that 50 percent or more of the available borings found soil layers that would liquify during a Magnitude 5.5 earthquake producing a peak ground acceleration of 0.10g. Another fourteen islands were concluded to have moderate liquefaction potential.

The authors state that it is not the intent of this report to predict the potential for levee failure, rather to identify levees within the Delta which are most susceptible to liquefaction damage. Also, that predicting degree of levee damage was beyond the scope of the study. This report provides useful summaries of boring data in the Delta, most of which were performed by the Department of Water Resources in the late 1950s. The simplified analysis predicts liquefaction for accelerations as low as 0.05g. However, historical information seems to indicate that failure would be unlikely at this low level of shaking. This is discussed further in Chapter 5. Amplification effects of the organic soils were not considered and ground response analyses were not performed.

3.9 EARTHQUAKE DAMAGE IN THE SACRAMENTO-SAN JOAQUIN DELTA
California Geology
February 1985

This report was written by Michael Finch and describes earthquake-induced levee damage for the 1906 earthquake and for five earthquakes which occurred between 1979 and 1984. These are the Coyote Lake, Livermore, Coalinga, Pittsburg, and Morgan Hill earthquakes. For these more recent earthquakes, 15 sites with earthquake related damage are reported (see Figure 3-8). Five

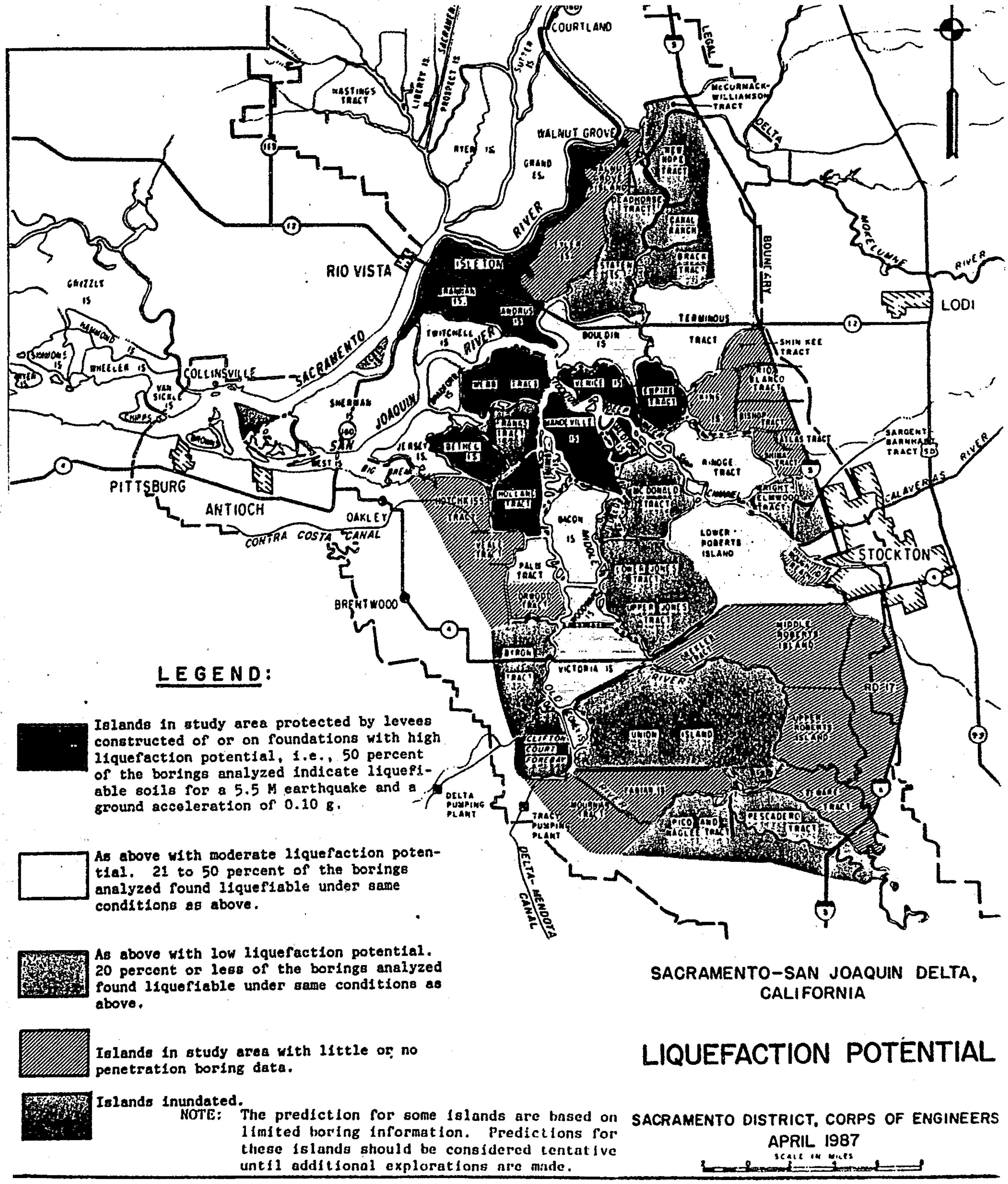


Figure 3-7: Liquefaction Potential, (from Corps of Engineers, 1987)

damage sites are on Webb Tract and six are on Venice Island. The other sites are on Mandeville Island, Bacon Island, Empire Tract, and King Island. These reports are based on interviews with Delta residents. There was no reported flooding as a consequence of this damage. The report also discusses the angular relationship between earthquake damaged levees and epicenters.

This is the only report concerning Delta levee damage due to earthquakes since the 1906 earthquake. Consequently, it is cited as a reference by most other reports on seismic evaluations of facilities in the Delta since 1985. This report concludes that Delta levees which contain large amounts of sand are most likely to be affected by earthquakes, particularly if the earthquake waves hit the levee broadside at an angle from 40 to 90 degrees.

These reports have been reinvestigated and the eyewitnesses have been re-interviewed. It has been found that the damage reported by Finch is, in most areas, difficult to definitely attribute to earthquake shaking (see Chapter 5).

3.10 MCDONALD ISLAND LEVEE STABILITY STUDY
Dames and Moore
January 1985

This presents the results of a study Dames and Moore undertook for PG&E to assist them in planning to reduce the potential risk of flooding of the Mcdonald Island facilities and/or to reduce the impact of flooding on their facilities. Only the chapter pertaining to the seismic evaluation is discussed here. An evaluation was made of the seismic risk at the McDonald Island site by using statistical procedures based on methods by Cornell and Vanmarcke (1969), and expanded by Donovan and Bornstein (1975).

Peak accelerations are 0.13g, with a probability on non-exceedance of 50 percent (called the operating level), and 0.18g, with a probability of non-exceedance of 95 percent (called the contingency level). The operating level represents the design life for the levees. The contingency level is that level of seismic activity at which the factor of safety of most of the levee area will approximately equal 1. It is reported that some isolated liquefaction might begin at accelerations less than 0.1g, 10 percent of the SPT samples were indicated as liquefiable at accelerations of 0.12 to 0.14g, and over 25 percent of the SPT samples were liquefiable at accelerations of 0.20 to 0.25g. It was concluded that liquefaction is not too likely for earthquakes with a recurrence interval of 50 years (0.11g).

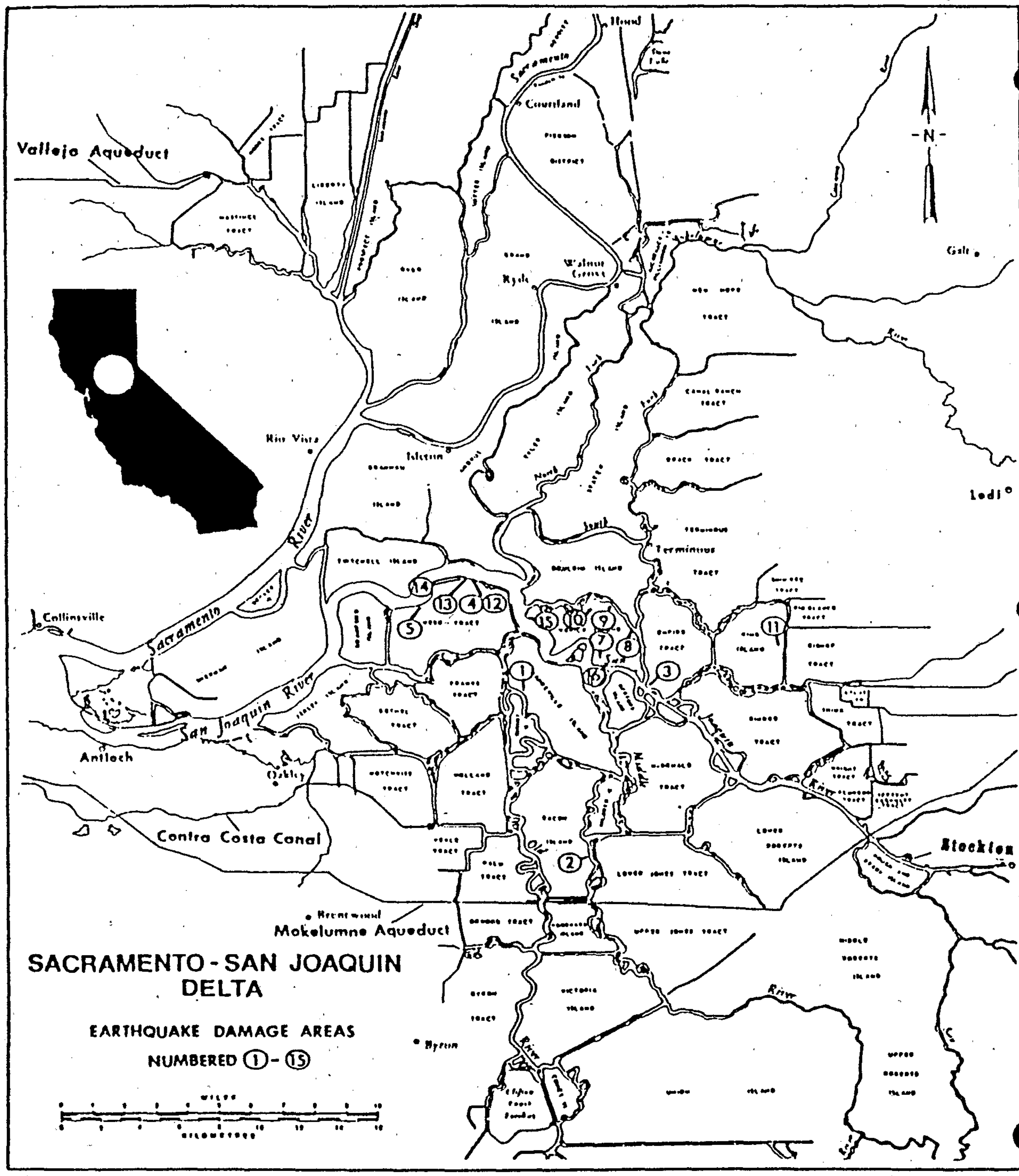


Figure 3-8: Ground Damage, (from Michael Finch).

3.11 SEISMICITY
California Department of Water Resources
Staff Paper
June 1984

This report presents a general view of the seismic vulnerability of Delta levees. It describes regional faulting and reports case histories of levee damage (those used in the California Geology paper discussed above). It also reviews other reports on seismicity in the Delta and gives general information about analytical procedures available for seismic evaluations. It presents a review of other reports related to Delta seismicity (see Table 3-1).

The study also reports the results of the analysis for the Mokelumne Aqueduct by Bolt (1977). This is a peak acceleration of 0.25g with a recurrence interval of 200 years and 0.20g with a recurrence interval of about 30 years. No analytical studies were performed, therefore, conclusions are of a general nature and state that levee failure may occur as a result of an earthquake.

3.12 DOCUMENTATION REPORT, SACRAMENTO-SAN JOAQUIN DELTA
Corps of Engineers, Sacramento
1982

This report presents data on topics such as hydrology of the Delta region, island subsidence, etc. Peak ground accelerations were based on curves developed by Schnabel and Seed (1972), tempered by the values recommended by Seed and Leeds for Success Dam and Terminous Dam (1980).

This report only briefly discusses earthquake related damage and states that failure may occur as a result of overstressing the soil or by liquefaction.

3.13 PARTIAL TECHNICAL BACKGROUND DATA FOR THE MOKELUMNE AQUEDUCT SECURITY PLAN
Converse Ward Davis Dixon
1980

The purpose of this report was to provide technical background data required for the Mokelumne Aqueduct Security Plan being prepared by the East Bay Municipal Utility District. Seismic evaluations are performed, including liquefaction analysis and seismic hazard determination. The seismic hazard evaluation employs probabilistic methods to determine relationships for return period and acceleration, and probability of exceedance and acceleration. This part of the report was supplemented with an addendum the following year. The addendum presented revised values for the relationships mentioned above. These results are depicted in Figure 3-9. These analyses show an acceleration of about 0.18g for a 43-year return period (30-year exposure time at 50 percent probability of non-exceedance).

The liquefaction analysis used corrected Standard Penetration Test results, as per Seed (1979). Detailed calculations of liquefaction potential were determined in another report and the results presented here. The results indicate that liquefaction will occur in one location at ground acceleration levels less than 0.1g, and at the other locations, ground accelerations of 0.1 to 0.2g are required.

3.14 SUMMARY

A general consensus among the authors is noticeable on some of the issues concerning earthquake evaluations of Delta levees. For example:

1. None of the reports could describe with certainty the amplification or attenuation effects of the Delta's organic soils. Some did not seem to address this issue at all.

2. Essentially all the reports state that liquefaction is likely to occur in the foundation soils beneath the organic soil layer. The reports find that in general the acceleration values required to trigger liquefaction are between 0.1 to 0.2g. There are fewer locations where liquefaction might occur at an acceleration value of less than 0.1g.

3. Larger acceleration values are anticipated in the southwestern portion of the Delta than in the northeastern part.

4. None of the studies report a past levee failure due to earthquake shaking.

5. Most of the authors recognize a need for additional data before a more conclusive answer regarding the vulnerability to earthquake shaking can be determined.

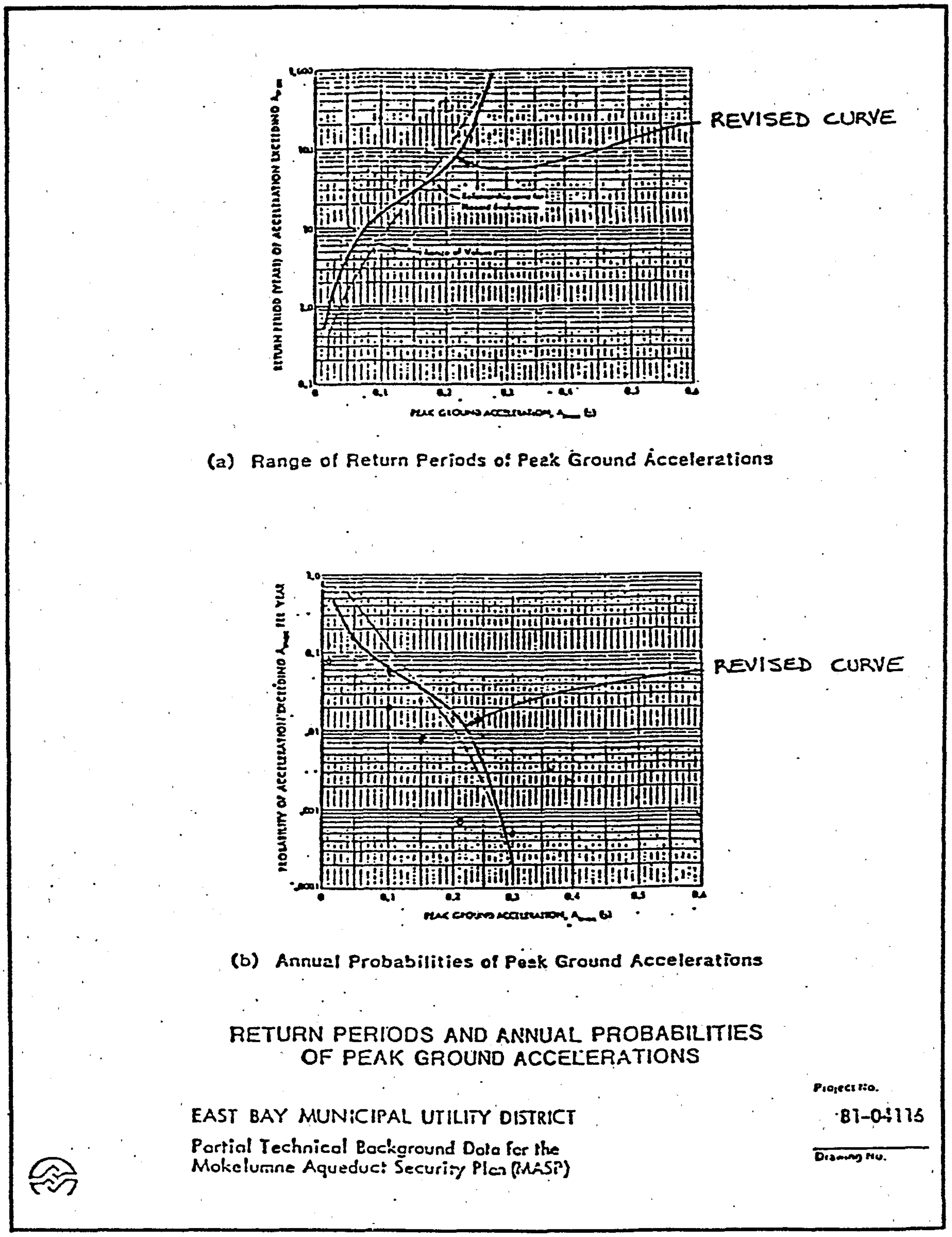


Figure 3-9: Seismicity of Project Area, (from Converse, Ward, Davis and Dixon, 1980).

C-072291

TABLE 3-1
REVIEW OF PERVIOUS REPORTS

Investigation Title and Author	TASK PERFORMED: Evaluation of Earthquake Sources	Estimate of Ground Accelerations	Liquefaction Analysis	Historical Evaluation of Reported Damage	Field Studies	Lab Testing	Dynamic Response Analysis	Seismic Risk Analysis	Deformation Analysis	Conclusions	Author's Recommendations	Comments on Investigation
Preliminary Seismic Risk Analysis, North Delta, Bureau of Reclamation, September 1991.	Describes regional faults, slip rates, and characteristic magnitude.	Estimates peak acceleration for the North Delta Region. 90% probability of non-exceedance in 100 years. Joyner and Boore, 1981 attenuation, and Seed-Idriss, 1982 attenuation.	SHAKE analysis and corrected SPT's.	None	Uses data from 1987 Corps of Engineers study and the 1987 Finch Master's Thesis.	None	SHAKE analysis using synthetic acceleration time history.	Computer program Seisrisk III presents contours of peak acceleration across North Delta.	Newmark-type deformation analysis using program DYNDSP.	For areas where peak accelerations are greater than 0.1, 23-36 percent of levees fail due to liquefaction.	Need more and better data to increase confidence.	Response analyses are not documented.
General Seismic and Geotechnical Risk Assessment, Sacramento-San Joaquin Delta, California, Dames and Moore, 1991.	Lists limiting magnitudes for regional faults.	Probabilistic approach develops peak acc. contours for Delta Region.	None	Cites data from the 1985 California Geology Report by Michael Finch.	None	None	None	None	None	Base ground motions expected to amplify in the Delta.	Lists recommendations for reducing vulnerability of levees, such as increase crest widths and flattened slopes.	Provides estimate of peak accelerations. Other information is very general.
Seismic Design Criteria, Wilkenson Dam, Bouldwin Island, California, Draft, -- Harding Lawson Associates, 1990.	Lists and describes regional faults and their MCE.	Based on Joyner and Boore 1981 attenuation. Based on MCE and probabilistic method for a return period of 500 years.	None	None	Shear wave velocity of organic soils.	Dynamic shear modulus and damping determined for peat samples.	None	Probabilistic approach yields 0.25g which is larger than 0.21g obtained by deterministic method.	None	None	Outlines recommended design approach.	Provides information on an approach for determining seismic stability of a proposed facility.

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TABLE 3-1
REVIEW OF PERVIOUS REPORTS

	TASK PERFORMED											
Investigation Title and Author	Evaluation of Earthquake Sources	Estimate of Ground Accelerations	Liquefaction Analysis	Historical Evaluation of Reported Damage	Field Studies	Lab Testing	Dynamic Response Analysis	Seismic Risk Analysis	Deformation Analysis	Conclusions	Author's Recommendations	Comments on Investigation
Preliminary Seismic Risk Analysis, South Delta -- Bureau of Reclamation, December 1989.	Describes regional faults, slip rates, and characteristic magnitudes.	Estimates peak acceleration values for two specified sites. 10% probability of exceedance in 100 years.	Response analysis using program Shake and corrected SPT's.	None	SPT data obtained from the 1985 Dames and Moore Study of McDonald Island.	None	Shake analysis using synthetic acceleration time history.	SEISRISK III yields 0.16g for Site 1 and 0.18g for Site 2. Joyner-Boore, 1981 attenuation relationship.	Newmark-type deformation analysis using Program DYNDSP.	20% of levees fail due to foundation liquefaction. 5-10% of levees fail due to levee liquefaction. Also, 10% by deformation.	Need more and better data to increase confidence.	Response analyses are not documented.
"Estimated Performance of Twitchell Island Levee System," Michael Finch, 1988.	Lists regional faults.	Estimates peak accelerations. Joyner and Boore attenuation equation.	Simplified analysis based on MCE and complete strength loss in silty sand.	None	None	None	None	None	Uses method prepared by Kutter. Calculates displacements up to 14,000 feet.	Concludes stabilizing berm will liquefy, deform, and result in levee failure.	Engineering solutions to the liquefaction problems in the Delta will require further study.	Analysis methods not clear.
Sacramento-San Joaquin Delta Levee Liquefaction Potential, USACE, 1987.	Describes regional faults.	Assumes 0.05, 0.10, and 0.2g.	Based on simplified procedures with corrected SPT's for most of Delta.	Cites data from the 1985 California Geology Report by Michael Finch.	Uses data from previous exploration programs.	None	None	None	None	Some levee failures will occur.	Need more and better date to increase confidence.	Good summary of available borehole data in the Delta. Assumes no amplification or attenuation from bedrock to ground surface.
"Seismicity" - DWR, March 1985 -- Preliminary, unpublished.	Lists regional faults.	None	States liquefaction may occur, no analysis.	Cites data included in 1985 California Geology Report by Michael Finch.	None	None	None	None	None	There are many uncertainties regarding seismic hazard to Delta.	Need more data to perform analysis.	Contains good general information.

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TABLE 3-1
REVIEW OF PERVIOUS REPORTS

	TASK PERFORMED											
Investigation Title and Author	Evaluation of Earthquake Sources	Estimate of Ground Accelerations	Liquefaction Analysis	Historical Evaluation of Reported Damage	Field Studies	Lab Testing	Dynamic Response Analysis	Seismic Risk Analysis	Deformation Analysis	Conclusions	Author's Recommendations	Comments on Investigation
McDonald Island Study, Levee Stability, Dames and Moore, 1985.	Describes regional faults.	Estimates peak ground acceleration from two cases. With Midland Fault and without Midland Fault by probabilistic method 50% probability of exceedance in 50 years. Yields 0.13g, 5% in 50 years yields 0.18g.	Two methods used: 1) compares historic liquefaction vs. seismic potential, and 2) simplified method proposed by Seed and corrected SPT's.	None	SPT data from borings at McDonald Island.	Not indicated	Rock motions obtained by probabilistic method.	Estimates peak acceleration based on: 1. Seismic zones with random occurrence, 2. Donovan and Bornstein attenuation, and 3. Give recurrence relation-ship.	None	Isolated liquefaction begins at <.1g. 10% of SPT indicates liquefactions at .1 to .14g 25% of SPT indicates liquefaction at 0.2 to 0.25g.	States that it would be difficult to improve the levees to assure that no liquefaction occurred. Also, peat is expected to behave similar to Bay Mud.	General assessment based on many simplifying assumptions; i.e., the surface motions are the same as the base rock motions.
Earthquake Damage in the Sacramento-San Joaquin Delta, California Geology, Mike Finch, 1985.	None	None	None	Reports levee damage caused by earth-quakes from 1979 to 1984.	Site inspections and interviews with witnesses.	None	None	None	None	Indicates angular relationship between damaged levees and epicenters. Predicts potential widespread damage even during moderate earthquake.	Does not give one.	Further investigation indicates that many of these case histories cannot be verified.
Documentation Report, Sacramento-San Joaquin Delta, California USACE, 1982.	Lists regional faults.	Based on Schnabel and Seed attenuation relationship.	None	None	None	None	None	States that insufficient data exists.	None	Liquefaction is a potential problem.	None related to seismic criteria.	Very little information presented on seismic considerations.

C—072293

TABLE 3-1
REVIEW OF PERVIOUS REPORTS

	TASK PERFORMED											
Investigation Title and Author	Evaluation of Earthquake Sources	Estimate of Ground Accelerations	Liquefaction Analysis	Historical Evaluation of Reported Damage	Field Studies	Lab Testing	Dynamic Response Analysis	Seismic Risk Analysis	Deformation Analysis	Conclusions	Author's Recommendations	Comments on Investigation
Mokelumne Aqueduct Security Plan, CWDD 1981, 1982.	Lists regional faults.	Probabilistic approach.	Uses simplified method and SPT.	None	Exploration and SPT Testing.	Classification and Density tests.	None	Probabilistic approach.	None	Isolated liquefaction occurs at 0.1g. 10% of SPT Tests indicated liquefaction at 0.1 to 0.2g.	Provides alternatives for improving stability of the aqueduct.	Provides useful data on site conditions and estimated peak acceleration.

4. ESTIMATES OF PEAK BEDROCK ACCELERATION

4.0 INTRODUCTION

Peak horizontal accelerations were estimated for rock beneath the Delta using two approaches. First, they were estimated by the deterministic method employing the Maximum Credible Earthquake magnitudes and the attenuation relationship developed by Idriss (1985) for rock and stiff soil sites. These values are representative of the accelerations that would occur at outcrops of the base rock layer beneath the softer alluvial and organic soils. In addition, these values were normalized for a Magnitude 7.5 event to aid in the liquefaction assessment of the Delta.

Secondly, peak horizontal accelerations were determined by the probabilistic method, often referred to as a risk analysis. This analysis uses the statistical method developed by Cornell and Vanmarcke (1969) with the values being computed using the computer program HAZARD developed by Idriss (1991). This method employed maximum magnitudes and the same attenuation relationship as was used in the deterministic method. These values were also normalized for a Magnitude 7.5 event.

4.1 DETERMINISTIC METHOD

For the current study using the deterministic approach, fault sources were assigned Maximum Credible Earthquake (MCE) values. The "Maximum Credible Earthquake" is defined by the California Division of Mines and Geology (1975) as the maximum earthquake that appears capable of occurring under the presently known tectonic framework. MCE values employed in this study are preliminary and require further study prior to a definite endorsement by DWR. To estimate peak accelerations for various distances, an attenuation curve appropriate for the earthquake event was used.

The attenuation curves used in this approach were developed by Idriss (1985) for rock and/or stiff soil outcrops and represent median values (see Figure 4-1). An additional refinement consisting of normalizing peak motions for the effect of different durations and magnitudes was also employed. Table 4-1 presents the fault sources, MCE values, and distances for various peak acceleration values used in this study.

Presented in Figures 4-2 through 4-9 are contours of median peak acceleration for potential rock and/or stiff soil outcrops in

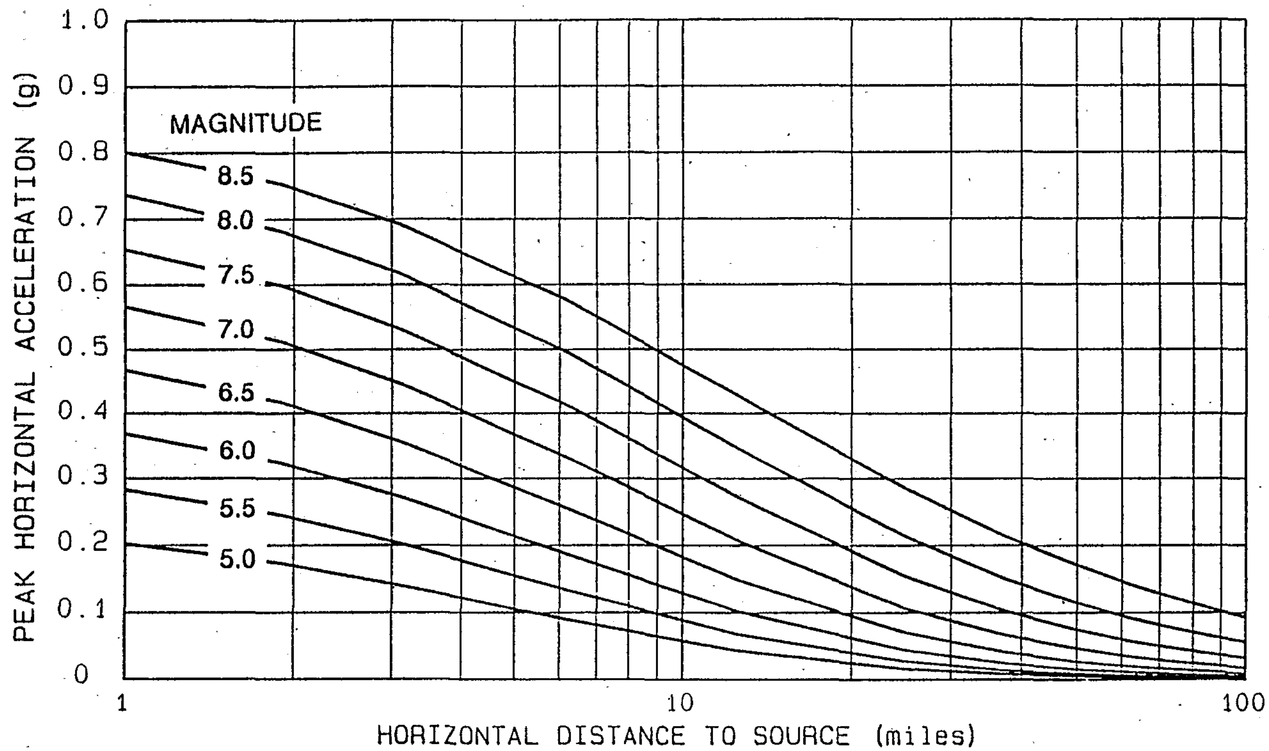


Figure 4-1: MEDIAN PEAK HORIZONTAL ACCELERATION VERSUS HORIZONTAL DISTANCE FOR DIFFERENT MAGNITUDES BASED ON IDRISS 1985 EQUATION FOR ROCK AND STIFF SOILS (VALUES OF ACCELERATION NORMALIZED TO M = 7.5)

Table 4-1: Predicted Values of Distance for Different Levels of Bedrock Acceleration (Values of Bedrock Acceleration Normalized to M=7.5)

FAULT	MCE MAGNITUDE	DISTANCE (miles)			
		0.05g	0.10g	0.15g	0.20g
SAN ANDREAS	8.5	147	73	47	33
SAN GREGORIO (SEAL COVE)	7.5	71	38	26	19
HAYWARD	7.5	71	38	26	19
CALAVERAS	7.3	56	31	21	15
GREENVILLE	6.5	33	19	12	9
GREEN VALLEY / CONCORD	6.5	33	19	12	9
COAST RANGE SIERRA NEVADA BOUNDARY ZONE	6.5	33	19	12	9
ANTIOCH FAULT SYSTEM	6.5	33	19	12	9

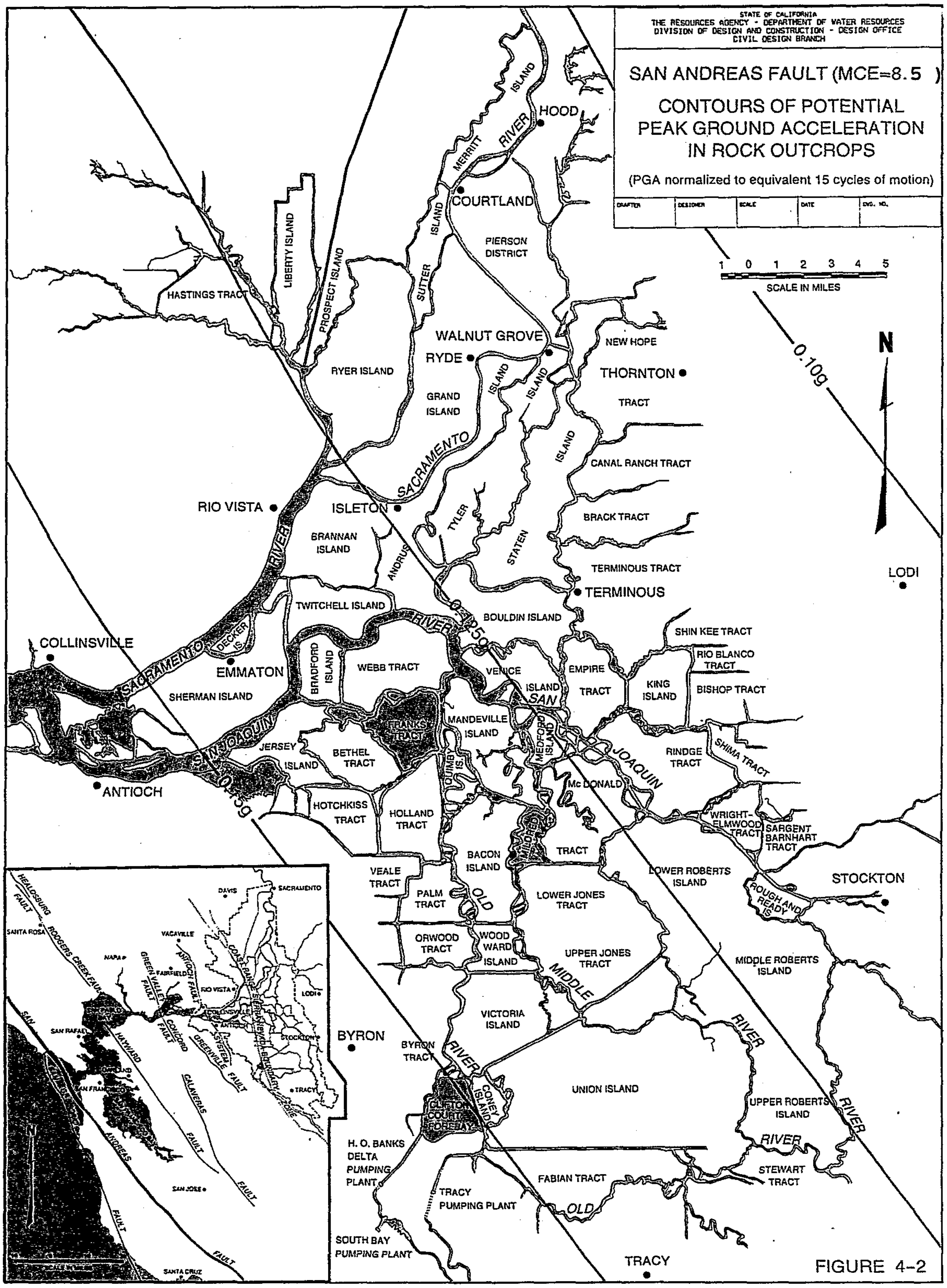

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
SAN ANDREAS FAULT (MCE=8.5)
CONTOURS OF POTENTIAL
PEAK GROUND ACCELERATION
IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
SCALE IN MILES
N
0.10g
0.125g
0.15g
HOOD
COURTLAND
PIERSON DISTRICT
MERRITT ISLAND
SUTTER ISLAND
LIBERTY ISLAND
PROSPECT ISLAND
HASTINGS TRACT
RYER ISLAND
WALNUT GROVE
RYDE
GRAND ISLAND
NEW HOPE
THORNTON
TRACT
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
TERMINOUS
LODI
SACRAMENTO
RIVER
RIO VISTA
ISLETON
BRANNAN ISLAND
ANDRUS
TYLER
STATEN
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
BISHOP TRACT
KING ISLAND
EMPIRE TRACT
VENICE ISLAND
COLLINSVILLE
EMMATON
DECKER IS.
SHERMAN ISLAND
BRADFORD ISLAND
WEBB TRACT
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
SAN JOAQUIN
RINDGE TRACT
SHIMA TRACT
JERSEY ISLAND
BETHEL TRACT
QUIMBY IS.
Mc DONALD
ANTIOCH
HOTCHKISS TRACT
HOLLAND TRACT
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
BACON ISLAND
MILDRED
TRACT
LOWER ROBERTS ISLAND
STOCKTON
ROUGH AND READY IS.
VEALE TRACT
PALM TRACT
OLD
LOWER JONES TRACT
ORWOOD TRACT
WOOD WARD ISLAND
UPPER JONES TRACT
MIDDLE
MIDDLE ROBERTS ISLAND
VICTORIA ISLAND
BYRON
BYRON TRACT
RIVER
UNION ISLAND
CLIFTON COURT FOREBAY
CONEY ISLAND
UPPER ROBERTS ISLAND
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
FABIAN TRACT
STEWART TRACT
SOUTH BAY PUMPING PLANT
TRACY
SACRAMENTO
DAVIS
VACAVILLE
SANTA ROSA
NAPA
FAIRFIELD
RIO VISTA
LODI
STOCKTON
TRACY
SAN RAFAEL
SAN FRANCISCO
SAN JOSE
SANTA CRUZ
HEALDSBURG FAULT
RODGERS CREEK FAULT
GREEN VALLEY FAULT
ANTIOCH FAULT
CONCORD FAULT
GREENVILLE FAULT
HAYWARD FAULT
CALAVERAS FAULT
SAN ANDREAS FAULT
COAST RANGE - SIERRAN BLOCK BOUNDARY ZONE

FIGURE 4-2

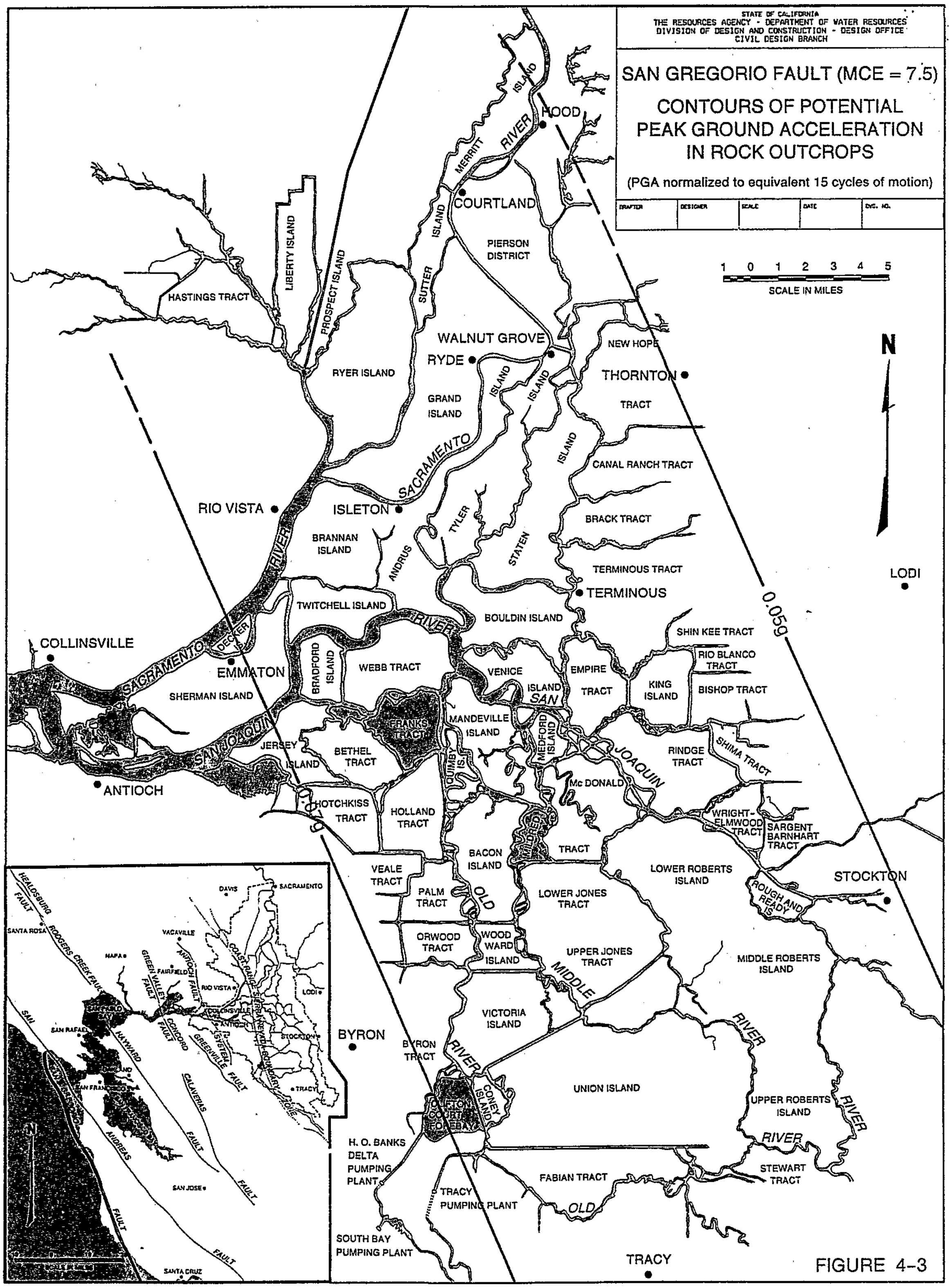

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
SAN GREGORIO FAULT (MCE = 7.5)
CONTOURS OF POTENTIAL
PEAK GROUND ACCELERATION
IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
1 0 1 2 3 4 5
SCALE IN MILES
N
0.059
HOOD
COURTLAND
PIERSON DISTRICT
WALNUT GROVE
RYDE
THORNTON
LODI
RIO VISTA
ISLETON
COLLINSVILLE
EMMATON
ANTIOCH
TERMINOUS
STOCKTON
BYRON
TRACY
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
RYER ISLAND
GRAND ISLAND
NEW HOPE TRACT
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
BRANNAN ISLAND
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
BISHOP TRACT
KING ISLAND
EMPIRE TRACT
VENICE ISLAND
WEBB TRACT
BRADFORD ISLAND
SHERMAN ISLAND
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
RINDGE TRACT
SHIMA TRACT
JERSEY ISLAND
BETHEL TRACT
Mc DONALD TRACT
HOTCHKISS TRACT
HOLLAND TRACT
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
BACON ISLAND
VEALE TRACT
PALM TRACT
LOWER JONES TRACT
LOWER ROBERTS ISLAND
ROUGH AND READY IS.
ORWOOD TRACT
WOODWARD ISLAND
UPPER JONES TRACT
MIDDLE ROBERTS ISLAND
VICTORIA ISLAND
BYRON TRACT
UNION ISLAND
CONEY ISLAND
CLIFTON COURT FOREBAY
UPPER ROBERTS ISLAND
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
FABIAN TRACT
STEWART TRACT
SOUTH BAY PUMPING PLANT
SACRAMENTO RIVER
SAN JOAQUIN
MIDDLE RIVER
OLD RIVER
DAVIS
SACRAMENTO
HEALDSBURG FAULT
RODGERS CREEK FAULT
SANTA ROSA
VACAVILLE
NAPA
FAIRFIELD
GREEN VALLEY FAULT
CONCORD FAULT
HAYWARD
CALAVERAS FAULT
SAN ANDREAS FAULT
SAN RAFAEL
SAN FRANCISCO
SAN JOSE
SANTA CRUZ
GREENVILLE FAULT
STOCKTON
TRACY
LODI
RIO VISTA
COLLINSVILLE

FIGURE 4-3

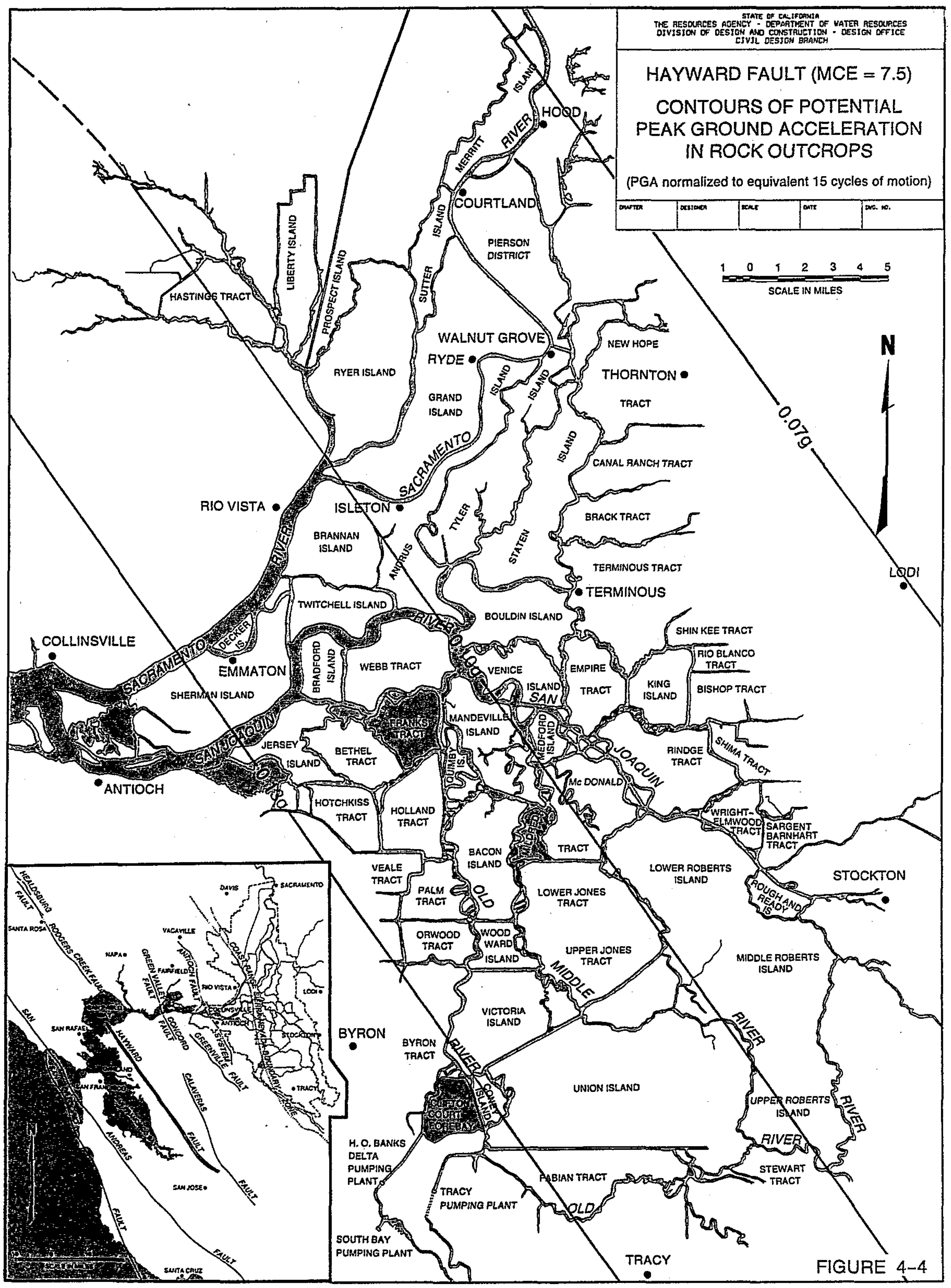
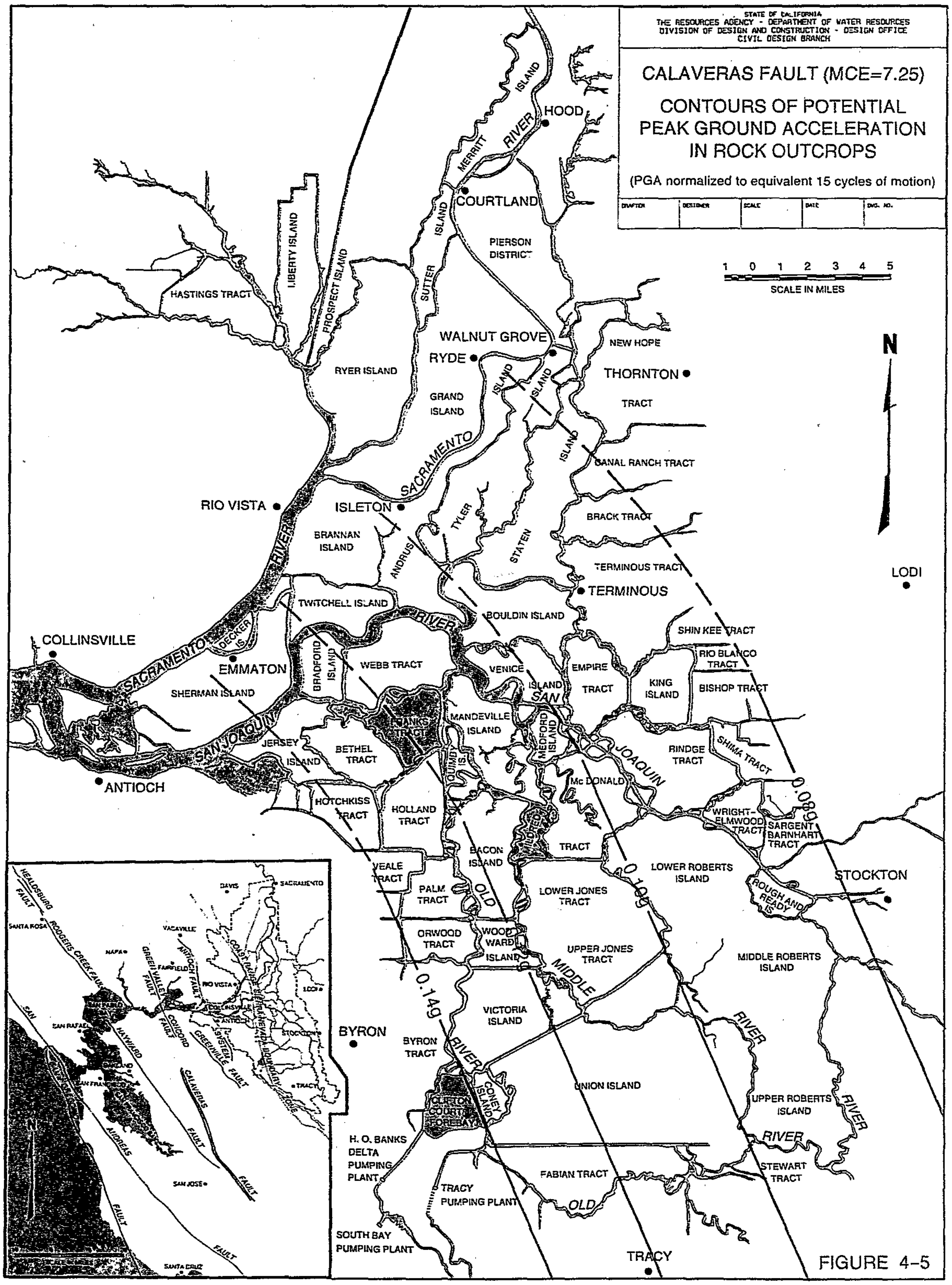

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
HAYWARD FAULT (MCE = 7.5)
CONTOURS OF POTENTIAL
PEAK GROUND ACCELERATION
IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
1 0 1 2 3 4 5
SCALE IN MILES
N
0.07g
HOOD
COURTLAND
PIERSON DISTRICT
WALNUT GROVE
RYDE
NEW HOPE
THORNTON
TRACT
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
TERMINOUS
LODI
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
SUTTER ISLAND
MERRITT ISLAND
RYER ISLAND
GRAND ISLAND
SACRAMENTO RIVER
RIO VISTA
ISLETON
BRANNAN ISLAND
ANDRUS
TYLER
STATEN
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
BISHOP TRACT
COLLINSVILLE
DECKER IS.
EMMATON
SHERMAN ISLAND
BRADFORD ISLAND
WEBB TRACT
VENICE ISLAND
EMPIRE TRACT
KING ISLAND
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
RINDGE TRACT
SHIMA TRACT
JERSEY ISLAND
BETHEL TRACT
QUIMBY IS.
Mc DONALD
SAN JOAQUIN
ANTIOCH
HOTCHKISS TRACT
HOLLAND TRACT
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
BACON ISLAND
TRACT
LOWER ROBERTS ISLAND
STOCKTON
VEALE TRACT
PALM TRACT
OLD
LOWER JONES TRACT
ROUGH AND READY IS.
ORWOOD TRACT
WOODWARD ISLAND
UPPER JONES TRACT
MIDDLE
MIDDLE ROBERTS ISLAND
VICTORIA ISLAND
BYRON
BYRON TRACT
RIVER
UNION ISLAND
UPPER ROBERTS ISLAND
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
FABIAN TRACT
STEWART TRACT
SOUTH BAY PUMPING PLANT
TRACY
HEALDSBURG FAULT
RODGERS CREEK FAULT
SANTA ROSA
NAPA
VACAVILLE
GREEN VALLEY FAULT
CONCORD FAULT
HAYWARD
CALAVERAS FAULT
GREENVILLE FAULT
SAN ANDREAS FAULT
SAN RAFAEL
SAN JOSE
SANTA CRUZ
DAVIS
SACRAMENTO
RIO VISTA
LODI
STOCKTON
TRACY

FIGURE 4-4



CALAVERAS FAULT (MCE=7.25)
CONTOURS OF POTENTIAL
PEAK GROUND ACCELERATION
IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)

DRAFTER	DESIGNER	SCALE	DATE	DWG. NO.

FIGURE 4-5

C — 0 7 2 3 0 1

C-072301

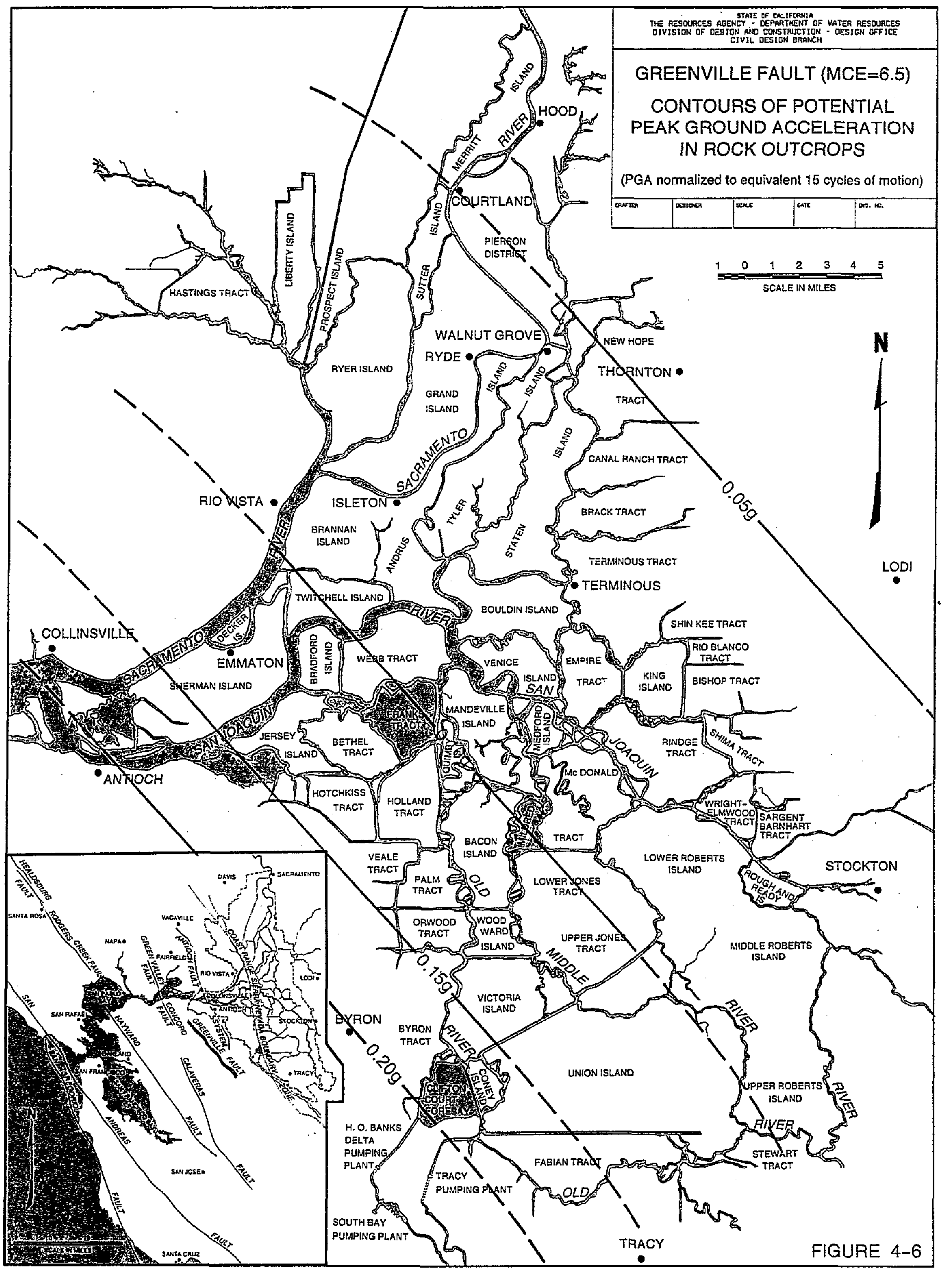
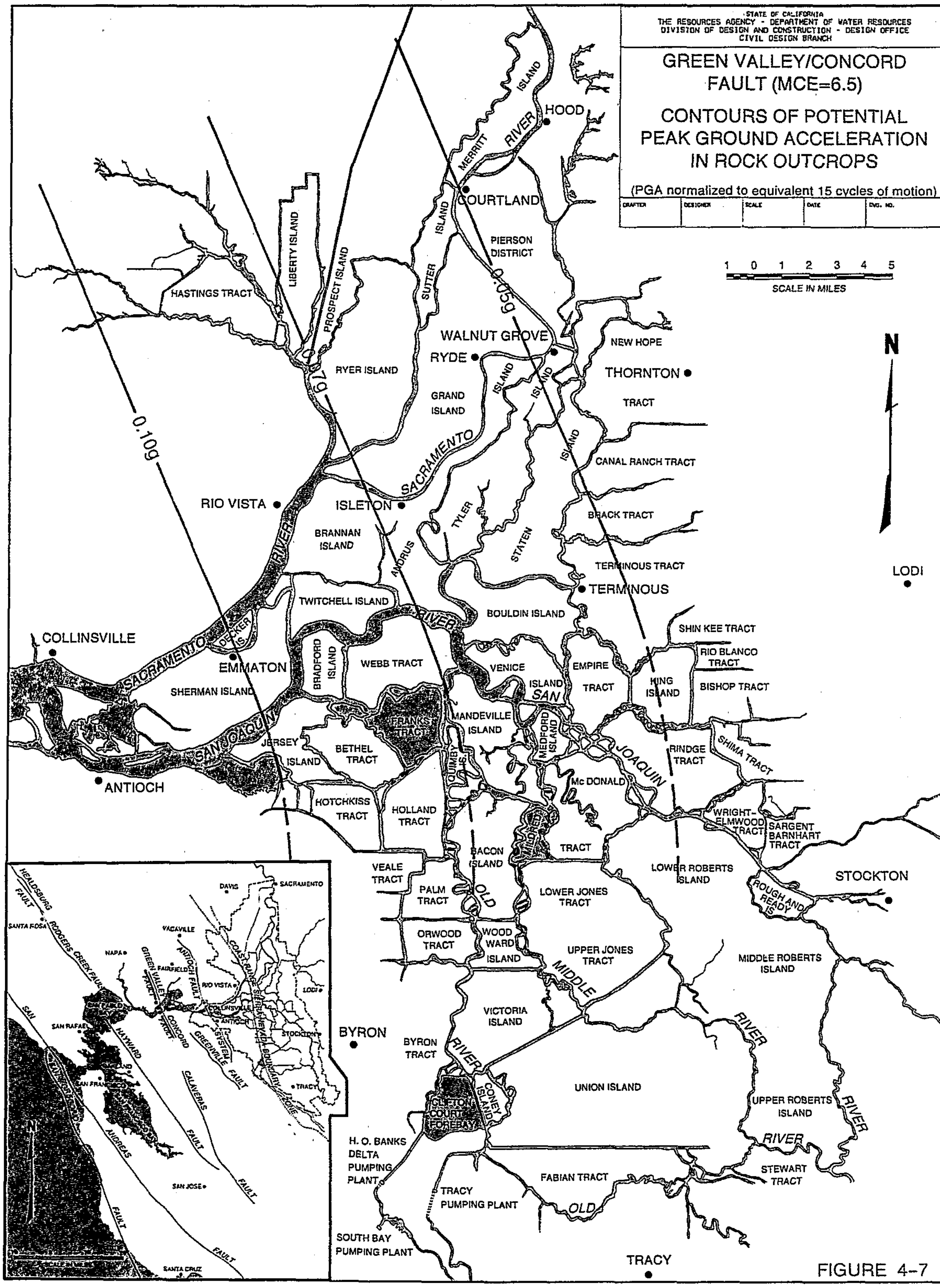

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
GREENVILLE FAULT (MCE=6.5)
CONTOURS OF POTENTIAL PEAK GROUND ACCELERATION IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
SCALE IN MILES
N
0.05g
0.15g
0.20g
HOOD
COURTLAND
WALNUT GROVE
RYDE
THORNTON
RIO VISTA
ISLETON
TERMINOUS
LODI
COLLINSVILLE
EMMATON
ANTIOCH
STOCKTON
BYRON
TRACY
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
RYER ISLAND
GRAND ISLAND
SUTTER ISLAND
MERRITT ISLAND
PIERSON DISTRICT
NEW HOPE TRACT
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
BRANNAN ISLAND
TYLER ISLAND
STATEN ISLAND
ANDRUS
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
BISHOP TRACT
KING ISLAND
EMPIRE TRACT
VENICE ISLAND
DECKER IS.
SHERMAN ISLAND
BRADFORD ISLAND
WEBB TRACT
FRANK TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
RINDGE TRACT
SHIMA TRACT
JERSEY ISLAND
BETHEL TRACT
McDONALD TRACT
HOTCHKISS TRACT
HOLLAND TRACT
QUIMBY
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
BACON ISLAND
LOWER ROBERTS ISLAND
VEALE TRACT
PALM TRACT
LOWER JONES TRACT
ROUGH AND READY IS.
ORWOOD TRACT
WOODWARD ISLAND
UPPER JONES TRACT
MIDDLE ROBERTS ISLAND
VICTORIA ISLAND
BYRON TRACT
UNION ISLAND
CLIFTON COURT FOREBAY
CONEY ISLAND
UPPER ROBERTS ISLAND
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
FABIAN TRACT
STEWART TRACT
SOUTH BAY PUMPING PLANT
SACRAMENTO RIVER
SAN JOAQUIN RIVER
OLD RIVER
MIDDLE RIVER
HEALDSBURG FAULT
RODGERS CREEK FAULT
GREEN VALLEY FAULT
ANTIOCH FAULT
COAST RANGE SIERRAN BLOCK BOUNDARY ZONE
CONCORD FAULT
GREENVILLE FAULT
SAN ANDREAS FAULT
HAYWARD
CALAVERAS FAULT
SANTA ROSA
NAPA
VACAVILLE
DAVIS
SACRAMENTO
FAIRFIELD
SAN RAFAEL
SAN FRANCISCO
SAN JOSE
SANTA CRUZ

FIGURE 4-6


STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
GREEN VALLEY/CONCORD FAULT (MCE=6.5)
CONTOURS OF POTENTIAL PEAK GROUND ACCELERATION IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
0.10g
0.07g
0.05g
SCALE IN MILES
N
HOOD
COURTLAND
WALNUT GROVE
RYDE
THORNTON
RIO VISTA
ISLETON
COLLINSVILLE
EMMATON
ANTIOCH
TERMINOUS
LODI
STOCKTON
BYRON
TRACY
SACRAMENTO RIVER
SAN JOAQUIN RIVER
MIDDLE RIVER
OLD RIVER
RYER ISLAND
GRAND ISLAND
BRANNAN ISLAND
TWITCHELL ISLAND
SHERMAN ISLAND
WEBB TRACT
BOULDIN ISLAND
FRANKS TRACT
BETHEL TRACT
JERSEY ISLAND
HOTCHKISS TRACT
HOLLAND TRACT
BACON ISLAND
MANDEVILLE ISLAND
VICTORIA ISLAND
UNION ISLAND
CLIFTON COURT FOREBAY
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
SOUTH BAY PUMPING PLANT
FABIAN TRACT
STEWART TRACT
UPPER ROBERTS ISLAND
MIDDLE ROBERTS ISLAND
LOWER ROBERTS ISLAND
UPPER JONES TRACT
LOWER JONES TRACT
KING ISLAND
BISHOP TRACT
RINDGE TRACT
SHIMA TRACT
EMPIRE TRACT
VENICE ISLAND
STATEN ISLAND
TYLER ISLAND
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
NEW HOPE TRACT
PIERSON DISTRICT
SUTTER ISLAND
MERRITT ISLAND
LIBERTY ISLAND
PROSPECT ISLAND
HASTINGS TRACT
SHIN KEE TRACT
RIO BLANCO TRACT
McDONALD TRACT
MEDFORD ISLAND
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
ROUGH AND READY IS.
VEALE TRACT
PALM TRACT
ORWOOD TRACT
WOODWARD ISLAND
BYRON TRACT
CONEY ISLAND
BRADFORD ISLAND
DECKER IS.
ANDRUS
QUIMBY IS.
FIGURE 4-7

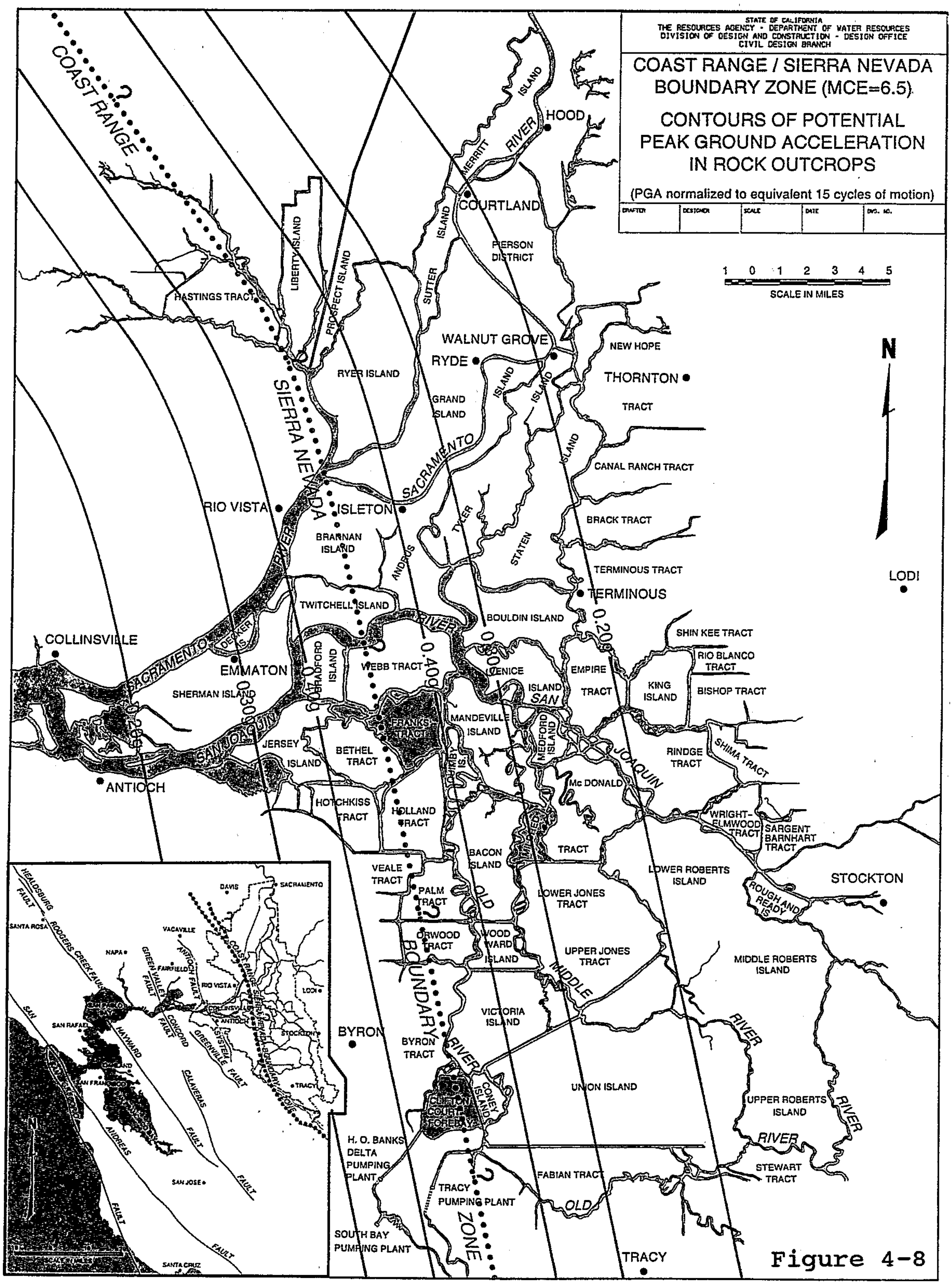

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
COAST RANGE / SIERRA NEVADA BOUNDARY ZONE (MCE=6.5)
CONTOURS OF POTENTIAL PEAK GROUND ACCELERATION IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
1 0 1 2 3 4 5
SCALE IN MILES
N
COAST RANGE
SIERRA NEVADA
BOUNDARY
ZONE
0.20
0.30
0.40
HOOD
COURTLAND
PIERSON DISTRICT
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
MERRITT ISLAND
SUTTER ISLAND
WALNUT GROVE
RYDE
RYER ISLAND
GRAND ISLAND
NEW HOPE
THORNTON
TRACT
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
TERMINOUS
LODI
RIO VISTA
ISLETON
BRANNAN ISLAND
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
COLLINSVILLE
EMMATON
SHERMAN ISLAND
WEBB TRACT
VENICE ISLAND
EMPIRE TRACT
KING ISLAND
BISHOP TRACT
JERSEY ISLAND
BETHEL TRACT
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
RINDGE TRACT
SHIMA TRACT
ANTIOCH
HOTCHKISS TRACT
HOLLAND TRACT
Mc DONALD
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
VEALE TRACT
PALM TRACT
BACON ISLAND
LOWER ROBERTS ISLAND
STOCKTON
ROUGH AND READY IS
LOWER JONES TRACT
ORWOOD TRACT
WOODWARD ISLAND
UPPER JONES TRACT
MIDDLE ROBERTS ISLAND
VICTORIA ISLAND
BYRON
BYRON TRACT
CLIFTON COURT FOREBAY
CONEY ISLAND
UNION ISLAND
UPPER ROBERTS ISLAND
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
FABIAN TRACT
STEWART TRACT
SOUTH BAY PUMPING PLANT
TRACY
SACRAMENTO RIVER
SAN JOAQUIN RIVER
OLD RIVER
MIDDLE RIVER
HEALDSBURG FAULT
RODGERS CREEK FAULT
SAN ANDREAS FAULT
HAYWARD FAULT
CALAVERAS FAULT
GREENVILLE FAULT
CONCORD FAULT
ANTIOCH FAULT
GREEN VALLEY FAULT
DAVIS
SACRAMENTO
SANTA ROSA
VACAVILLE
NAPA
FAIRFIELD
RIO VISTA
LODI
STOCKTON
SAN RAFAEL
TRACY
SAN JOSE
SANTA CRUZ

Figure 4-8

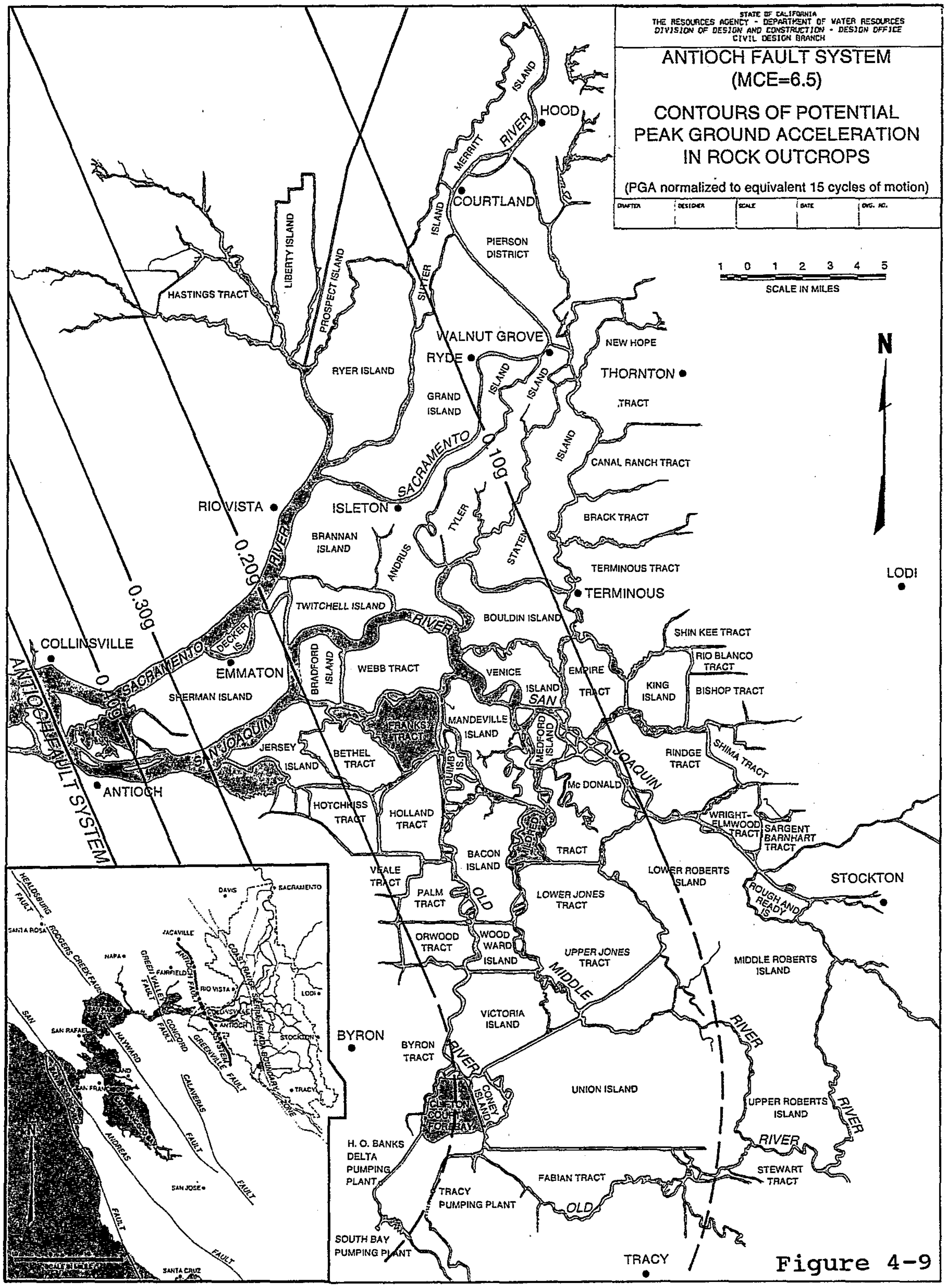

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
ANTIOCH FAULT SYSTEM
(MCE=6.5)
CONTOURS OF POTENTIAL
PEAK GROUND ACCELERATION
IN ROCK OUTCROPS
(PGA normalized to equivalent 15 cycles of motion)
SCALE IN MILES
N
0.30g
0.20g
0.10g
ANTIOCH FAULT SYSTEM
HOOD
COURTLAND
WALNUT GROVE
RYDE
THORNTON
RIO VISTA
ISLETON
TERMINOUS
LODI
COLLINSVILLE
EMMATON
ANTIOCH
STOCKTON
BYRON
TRACY
SACRAMENTO
SAN JOAQUIN
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
RYER ISLAND
GRAND ISLAND
PIERSON DISTRICT
BRANNAN ISLAND
TWITCHELL ISLAND
SHERMAN ISLAND
BOULDIN ISLAND
WEBB TRACT
VENICE ISLAND
MANDEVILLE ISLAND
BETHEL TRACT
JERSEY ISLAND
HOTCHKISS TRACT
HOLLAND TRACT
BACON ISLAND
PALM TRACT
ORWOOD TRACT
VICTORIA ISLAND
UNION ISLAND
FABIAN TRACT
LOWER JONES TRACT
UPPER JONES TRACT
LOWER ROBERTS ISLAND
MIDDLE ROBERTS ISLAND
UPPER ROBERTS ISLAND
STEWART TRACT
KING ISLAND
BISHOP TRACT
RINDGE TRACT
SHIMA TRACT
SHIN KEE TRACT
RIO BLANCO TRACT
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
NEW HOPE TRACT
EMPIRE TRACT
Mc DONALD TRACT
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
SOUTH BAY PUMPING PLANT
BYRON TRACT
CLIFTON COURT FOREBAY
OLD RIVER
MIDDLE RIVER

Figure 4-9

the Delta area. As may have been deduced by the fact that most of the fault sources considered are to the west, higher acceleration values are generally predicted for the western part of the Delta, with smaller acceleration values being calculated for the eastern side. The exception to this is for the postulated Coast Range-Sierra Nevada Boundary Zone shown in Figure 4-9 which shows relatively high accelerations centered in the central portion of the Delta. It should be noted that no one earthquake can produce contours with such a large envelope of potential levels of acceleration. Because it is assumed that the MCE can occur on any segment of the fault in question, the contours presented simply represent an envelope of the potential median levels of acceleration that the various fault zones can produce in base materials.

Table 4-2 presents potential peak accelerations for rock and/or stiff soil outcrops at three locations in the Delta (Sherman Island, Terminous Tract, and Old River; see Figure 4-10 for their locations). These locations were selected because they cover a large range in geographic locations. Most of the acceleration levels are relatively small with typical values generally between 0.05g and 0.15g. The highest peak acceleration value shown, 0.32g at Old River, results from a Magnitude 6.5 event on the postulated Coast Range-Sierra Nevada Boundary Zone.

4.2 PROBABILISTIC METHOD

A probabilistic method of analysis was performed to estimate probable base accelerations for different exposure periods and/or annual return periods. A computer program developed by DWR's consultant in earthquake engineering, Dr. I. M. Idriss, was used as the principal tool for the investigation. HAZARD (Idriss, 1991) is a computer program that performs a probabilistic seismic-hazard evaluation based on Gutenberg and Richter's 1954 frequency-magnitude relationship of earthquake occurrence. It is designed to provide acceleration levels from earthquake-induced ground shaking, as well as annual probabilities and return periods of peak accelerations. HAZARD calculates the individual and combined contributions from all pertinent seismic sources (faults). Idriss (1985) curves for rock and/or stiff soil conditions were used to model the attenuation of acceleration with distance.

Several inputs (physical and statistical parameters) are required of each seismic source that might affect the study site, in order to run the HAZARD program. These include fault geometry, slip rate, distance from the site, maximum-magnitude earthquake, and slope "b" of the earthquake recurrence relationship (magnitude vs. cumulative number of events per year).

Table 4-2: Predicted Values of Peak Horizontal Bedrock Acceleration at Sherman Island, Terminous Tract, and Old River Crossing (Values of Acceleration Normalized to M=7.5)

FAULT	MCE MAGNITUDE	SHERMAN ISLAND DISTANCE (miles)	SHERMAN ISLAND ACCEL. (g)	TERMINOUS TRACT DISTANCE (miles)	TERMINOUS TRACT ACCEL. (g)	OLD RIVER CROSSING DISTANCE (miles)	OLD RIVER CROSSING ACCEL. (g)
SAN ANDREAS	8.5	50	0.14	65	0.11	51	0.14
SAN GREGORIO (SEAL COVE)	7.5	55	0.07	70	0.05	50	0.06
HAYWARD	7.5	30	0.13	46	0.08	34	0.11
CALAVERAS	7.3	21	0.15	35	0.09	25	0.13
GREENVILLE	6.5	13	0.14	27	0.06	16	0.12
GREEN VALLEY / CONCORD	6.5	18	0.10	34	0.05	25	0.07
COAST RANGE SIERRA NEVADA BOUNDARY ZONE	6.5	5	0.29	12	0.16	4	0.32
ANTIOCH FAULT SYSTEM	6.5	5	0.29	21	0.09	13	0.14

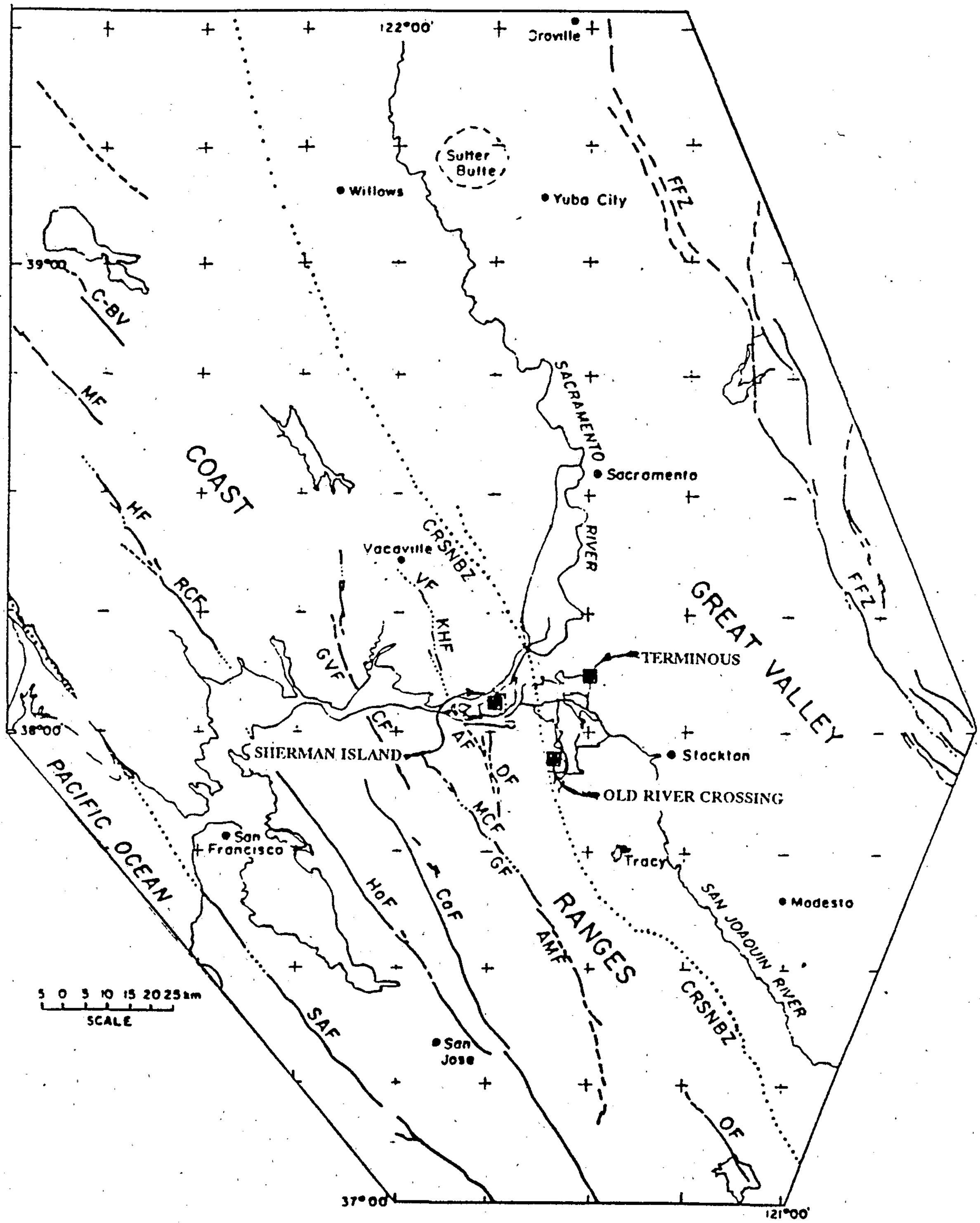


Figure 4-10: Location of Three Delta Sites and Local Faults (From Ake, et al, 1991)

The DWR sites at Terminous, Old River Crossing, and Sherman Island were chosen for this seismic-hazard assessment because they are representative of eastern, southern, and western locations in the Delta.

4.3 DISCUSSION OF EARTHQUAKE MAGNITUDE SCALES

Earthquake magnitudes for sources within this study's area of investigation are given in terms of the following magnitude scales: for magnitudes less than 7, local magnitude (M_L) is used; for magnitudes 7 and greater, surface-wave magnitude (M_s) is used. The reason for this is that the local-magnitude scales have a limiting value, or "saturation level" at which their usefulness ceases with increasing magnitude. This is consistent with previous assessments performed by Seed and Idriss (1982) and Idriss (1985). For simplicity, seismic events within this study, measured on whichever scale, are all given the magnitude designation "M".

4.4 MAXIMUM MAGNITUDE EARTHQUAKES AS USED IN THIS STUDY

It was necessary to assess maximum earthquake magnitudes for all seismic sources in this investigation in order to perform the seismic-hazard analysis.

Idriss (1991b) states:

> "Maximum earthquake magnitude on a fault is related to source geometry, fault behavior, and historical seismicity. Several empirical and analytical relationships have been developed to estimate maximum magnitudes (e.g., Slemmons, 1982; Schwartz et al., 1984). These relationships are based on the association between magnitude and (1) fault rupture length, (2) fault rupture area, (3) maximum displacement per event, (4) fault slip rate, or (5) seismic moment. Each of these relationships are subject to some uncertainty. The historical seismicity record may provide information on maximum magnitudes, particularly in cases where the historical record is quite long and/or the rate of activity on a fault is high."

Idriss (1985) summarizes:

> "Selection of a maximum magnitude for each source is ultimately a judgment that incorporates understanding of specific fault characteristics, the regional tectonic environment, similarity to other faults in the region, and data on regional seismicity."

For the purposes of the current investigation, maximum magnitudes are equivalent to MCE values.

4.5 PREVIOUS STUDIES

The approach of the current study is similar to seismic risk analyses performed in other investigations (e.g., Ostenaa, et al., 1989; Dames and Moore, 1991; Harding-Lawson and Associates, 1991, and Ake, et al., 1991). The parameters employed in the current study are probably closest to those employed by the USBR in their seismic risk analyses for North and South Delta facilities. In those studies, computer Program SEISRISK III was employed.

SEISRISK III is a probabilistic seismic-hazard assessment program. It integrates relevant earthquake sources in a region and yields horizontal acceleration or velocity values for any specified exposure period and probability of non-exceedance. The program requires a geographic grid describing locations of fault sources and sites of interest for which calculations are performed.

The USBR assessments and current study cover much of the same general study area and, consequently, the same fault sources. As a result, much of the input concerning fault sources was similar. Differences between the two studies include:

1. The current study incorporated a range in potential earthquake magnitudes for the different fault sources. This range was as low as Magnitude 5 and as high as the MCE. This is different than the characteristic magnitude approach used by the USBR.

2. The current study assumed an MCE value of 8.5 for the San Andreas Fault rather than the 8.0 characteristic earthquake employed by the USBR.

3. The theoretical CRSNB, based on updated information from Wong (1991) was treated in the current study as three contiguous segments, each with its own estimated slip rate and MCE (see Table 4-3).

Table 4-3: Principal Earthquake Sources

Fault or source	Maximum Magnitude	Slip Rate (mm/yr)	References*
San Andreas	8.5	19	1, 7, 8
Hayward	7.5	9	1, 3, 6, 7, 8
Calaveras	7.25	7.5	1, 3, 6
Healdsburg - Rodgers Creek	7	9	1, 8
Maacama	7.5	7	1, 4, 9
Green Valley - Cordelia	6.5	4	1, 3
Concord	6.5	4	1, 3
Marsh Creek	6.5	0.1 to 1.0 **	1, 3, 10
Greenville	6.5	0.1 to 1.0**	1, 3, 10
Arroyo Mocho	6.5	0.1 to 1.0**	1, 3, 10
Vaca	6	.02 to 0.1**	1, 2, 9, 10
Kirby Hill	6	.02 to 0.1**	1, 10
Antioch	6.5	0.3	1, 10
Davis	6	0.1	1, 10
Foothills	6.5	0.007	1, 5, 10
CRSNB, Segment A	6.8	0.5 to 1.0**	11
CRSNB, Segment B	6.5	0.1	11
CRSNB, Segment C	6.5	0.5	11
Ortigalita	6.75	0.1 to 1.0**	1, 10

* Full titles are listed in "References Cited" section.

** The highest value of each range was used in this study.

1. Ake and others, 1991
2. Clark and others, 1984
3. ESA , 1982
4. Herd , 1979
5. Jennings, 1975
6. Toppozada and others, 1981
7. USGS, 1988
8. USGS, 1990
9. Wesnousky, 1986
10. Wong, verbal communication, 1991
11. Wong, written communication, 1991

4. The current study employed seismic sources from 19 known or postulated fault sources and did not employ a "random" or background seismic source.

5. The acceleration attenuation relationships used in the current study were those developed by Idriss (1985) for rock and/or stiff soil. The USBR studies had used relationships developed by Joyner and Boore (1981). In addition, the current study normalized the acceleration values for the effects of different durations associated with different magnitudes.

6. Greater segment lengths for the San Andreas Fault and Foothill Fault Zone, extending well beyond the boundaries of the USBR study area, were used in the current study. This is due to basic differences in input requirements for Programs SEISRISK III and HAZARD.

4.6 METHODOLOGY

A key input required to ultimately run HAZARD is the "b" value or slope of the earthquake recurrence relationship for each seismic source. This is based on the record of seismic events that have historically occurred along a particular fault. For the San Andreas Fault the "b" values used by Idriss (1989) in his sample input file for the HAZARD program were used per his suggestion (Idriss, verbal communication, 1991). However, the data are too sparse to determine "b" for other individual faults; so for these faults a listing of historical seismic events was obtained from DWR's Earthquake Engineering Branch (see Figure 4-11). Using these historical data, an areal recurrence relationship (magnitude vs. cumulative events per year) was plotted. A best-fit line was determined, providing the slope "b" required to ultimately run HAZARD. The resulting value was 0.785, very close to the average "b" value of 0.8 for earthquake recurrence in California (Wong, verbal communication, 1991).

It should be noted that the cumulative occurrence vs. magnitude plot for the San Andreas Fault shows a change in the "b" value for earthquakes greater than M = 6.5. This is in contrast to the linear approach, e.g., the assignment of the previously-mentioned single "b" value, taken with the other faults studied here.

The analysis proceeded by inputing "b" values and other pertinent data for each source (i.e., fault geometry, slip rate, and maximum-magnitude event) into a spreadsheet program for each fault. The key output produced from this program was "ALPHA", the

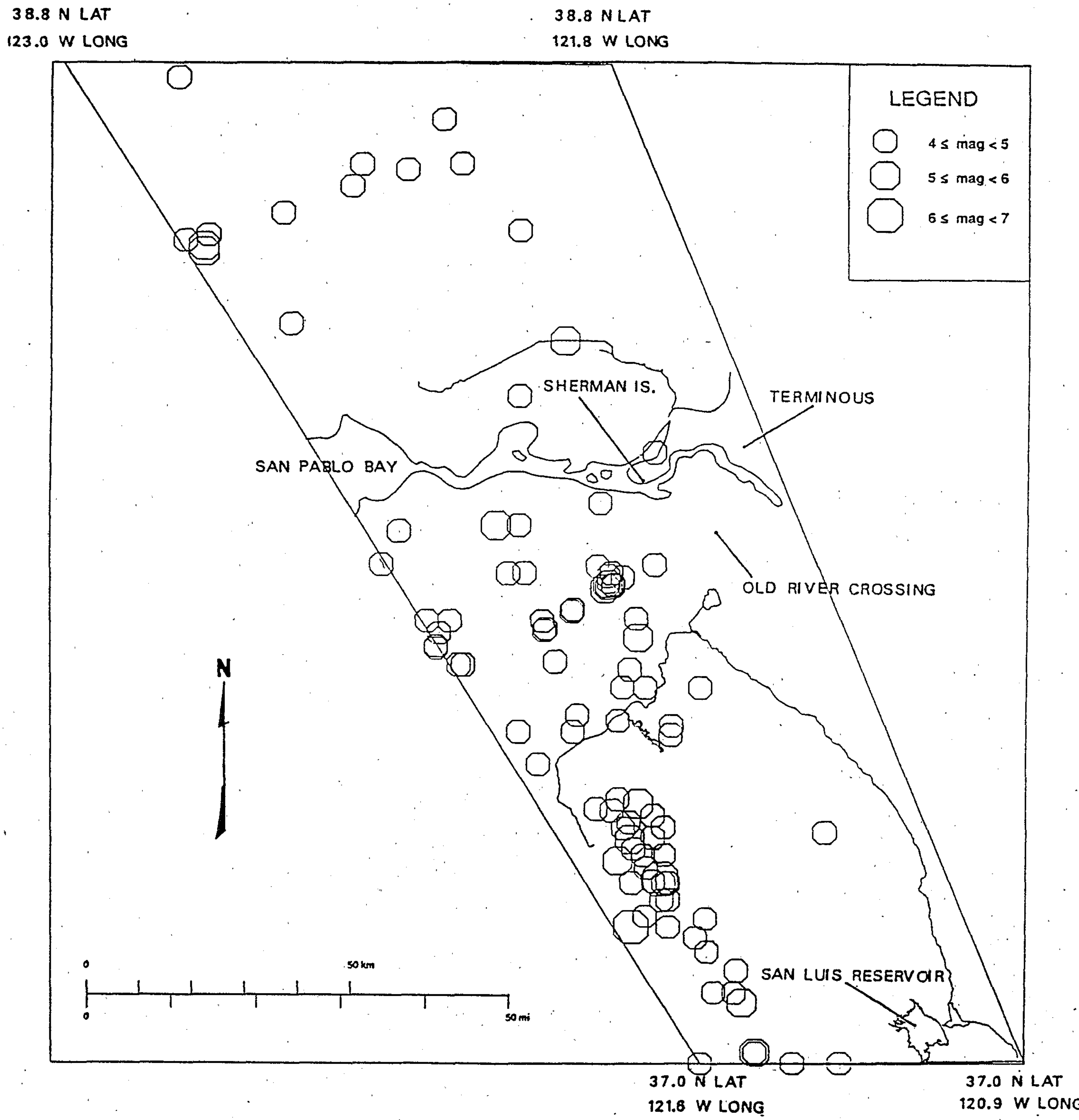


Figure 4-11: Selected Area of Historical Seismicity Showing Epicenters of Events Between M=4 and M=7, 1900-1991

number of events per year of the minimum magnitude of interest for magnitudes between 4.5 and the maximum magnitude.

With "ALPHA" and other pertinent data for each fault as input, a new input file, which was to become the raw material with which to run HAZARD, was created for each of the three Delta sites (see Appendices A7, A8, and A9).

In addition to the hazard analysis performed using the above-mentioned input files, HAZARD was also run incorporating a "magnitude weighing factor" for liquefaction, to aid in a liquefaction analysis of the Delta sites. This has the effect of normalizing the attenuation relationship to a magnitude M = 7.5 event, with respect to liquefaction. These results appear in Appendices A4, A5, and A6.

Table 4-4 and Figures 4-12 through 4-14 show the percentage that each fault system contributes to the ground acceleration at the Sherman Island, Old River, and Terminous sites for accelerations of 0.1g, 0.2g, and 0.3g. Note that these values are for magnitude weighing (M = 7.5) with respect to liquefaction.

4.7 EARTHQUAKE SOURCES, MCE's, AND SLIP RATES

Ake, et al. (1991), whose study employed characteristic magnitudes for seismic sources, relied heavily on the compilations of Wesnousky (1986) and Clark, et al. (1984) for basic fault behavior in the region, and used USGS (1988) as a primary source of information for the San Andreas and Hayward Faults (see Chapter 2). For this report, updated slip-rate estimates for several faults were obtained from USGS (1990) for the San Andreas, Hayward, and Healdsburg-Rodgers Creek Faults and from Wong (written and verbal communication, 1991) for the Marsh Creek, Greenville, Arroyo Mocho, Vaca, Kirby Hill, Antioch, and Davis Faults, and the CRSNB. These values are listed in Table 4-3.

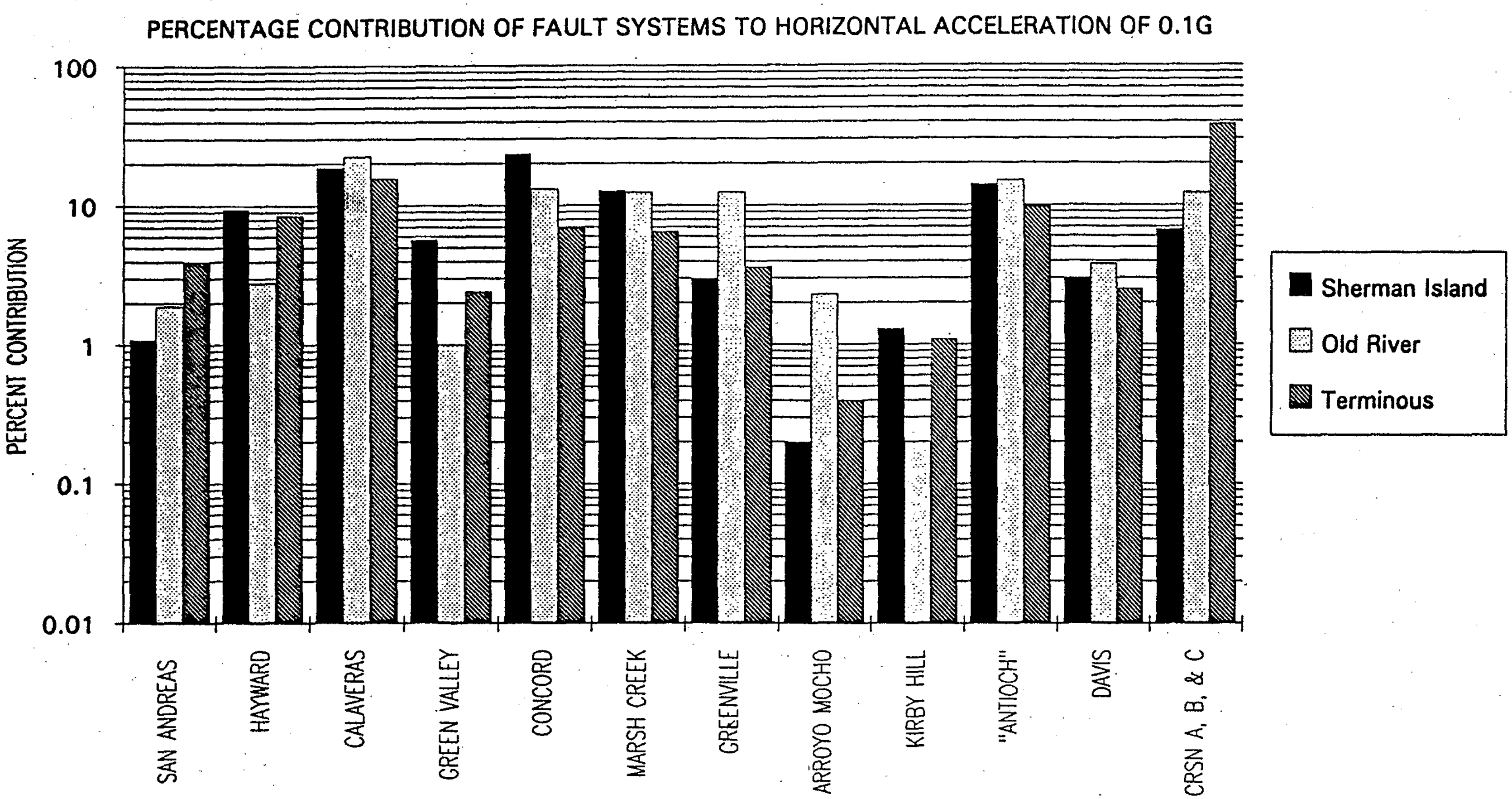
DWR does not necessarily endorse all the values for fault dynamics or behavior, or the physical or dimensional characteristics of the fault systems that were used as earthquake sources in this investigation. DWR has not independently evaluated the tectonic activity of these fault systems. However, based on the fact that this is a preliminary investigation, the estimates and judgments of fault behavior and fault characteristics that were obtained from the above various sources are assumed to be appropriate for this study. The evaluations performed by others are used only to obtain preliminary results.

Table 4-4: Percentage Contributions of Fault Systems to Horizontal Acceleration

SEISMIC STABILITY OF DELTA LEVEES
PERCENTAGE CONTRIBUTIONS OF FAULT SYSTEMS TO HORIZONTAL ACCELERATION

FAULT	HORIZONTAL ACCELERATION = 0.1G			HORIZONTAL ACCELERATION = 0.2G			HORIZONTAL ACCELERATION = 0.3G		
SYSTEMS	Sherman Island	Old River	Terminous	Sherman Island	Old River	Terminous	Sherman Island	Old River	Terminous
SAN ANDREAS	1.1	1.9	4	1	2	3.8	0.5	1.1	1.6
HAYWARD	9.5	2.8	8.5	2.8	0.3	1.2	0.5	0	0
CALAVERAS	18.8	22.5	15.7	10	9.1	3.3	3.7	1.9	0
GREEN VALLEY	5.7	1	2.4	1.5	0	0	0.2	0	0
CONCORD	23.4	13.1	6.9	15.7	5.3	0.3	7.7	0.9	0
MARSH CREEK	12.8	12.4	6.5	16	13.8	2.8	14.3	11.3	0
GREENVILLE	3	12.4	3.6	1.8	13.8	1.2	0.9	11.3	0
ARROYO MOCHO	0.2	2.3	0.4	0	0.8	0	0	0.2	0
KIRBY HILL	1.3	0.2	1.1	1.4	0.1	0.5	1.3	0	0.1
"ANTIOCH"	14.1	15.1	9.8	35	32.4	10.2	54.2	48.1	6.2
DAVIS	3	3.8	2.5	5.1	5.6	2.2	6.2	6.4	1.3
CRSN A, B, & C	6.7	12.3	38	9.5	16.9	74.3	10.6	18.8	90.7
OTHER	0.3	0.1	0.6	0	0	0.1	0	0.2	0

Percentages that each fault system contributes to horizontal accelerations of 0.1G, 0.2G, and 0.3G at the Sherman Island, Old River, and Terminous sites. Percentages are based upon magnitude weighting (M = 7.5) with respect to liquefaction.



Note: Acceleration value normalized to M = 7.5

Figure 4-12: Percentage Contribution of Fault Systems to Horizontal Acceleration of 0.10g

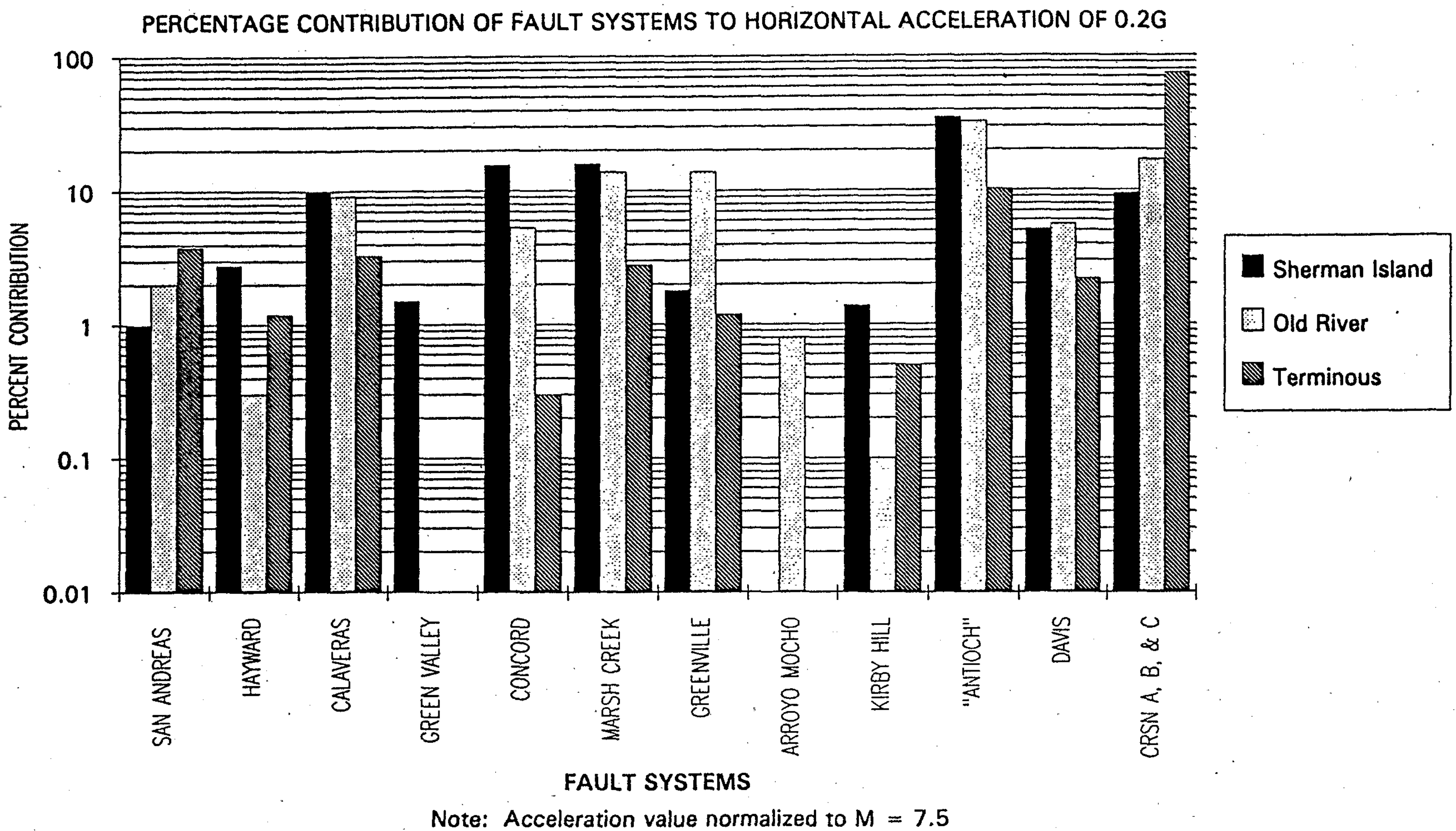
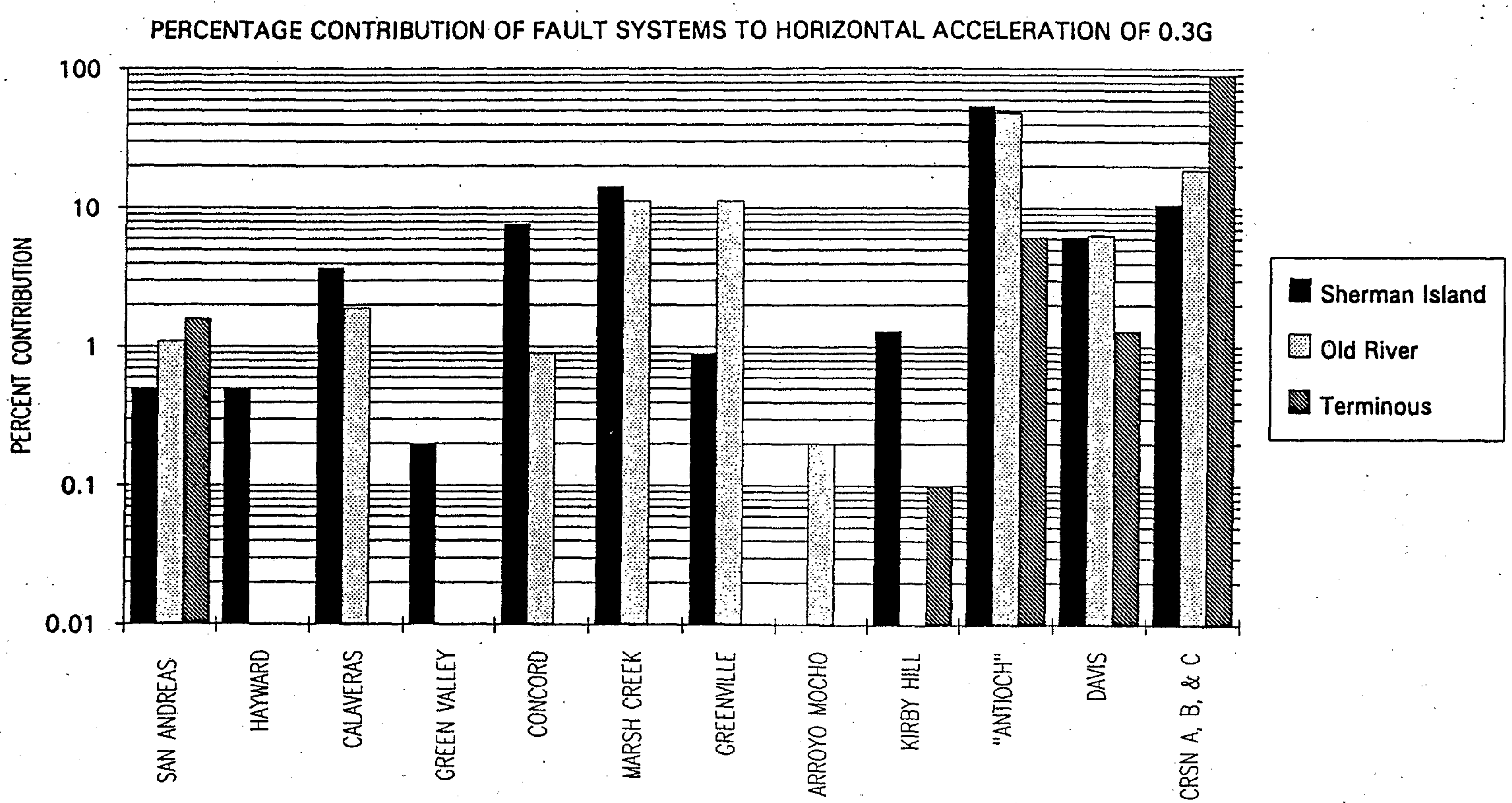


Figure 4-13: Percentage of Contributions of Fault System to Horizontal Acceleration of 0.20g



Note: Acceleration value normalized to M = 7.5

Figure 4-14: Percentage of Contribution of Fault System to Horizontal Acceleration of 0.30g

4.8 VARIABLE SLIP RATES

Wong's updated slip-rate data (see Table 4-3) show ranges for slip-rate estimates for six sources within the study area: the Marsh Creek-Greenville, the Arroyo Mocho, Vaca, and Kirby Hill Faults, as well as Segment A of the CRSNB. In this preliminary DWR analysis, a conservative analytical approach was taken (i.e., the high end of each slip-rate range, producing the scenario of a higher-magnitude earthquake, was used as input to run HAZARD).

4.9 RESULTS

Predicted bedrock accelerations, as well as annual probabilities and return periods of peak and selected accelerations for the DWR sites at Terminous, Old River Crossing, and Sherman Island, are given in Tables 4-5 and 4-6, Figures 4-15 and 4-16, and Appendices A1 through A6. In order to provide information relating to other locations in the Delta, the results for the three sites were used to develop contours of peak bedrock acceleration in Figures 4-17 and 4-18. The shapes of the contours were influenced in part by the results of the deterministic studies, as well as supplemental analyses performed in-house using SEISRISK III.

4.10 COMPARISON WITH OTHER STUDIES

The values obtained by the probabilistic method in this study were compared with similar analyses conducted by others (see Tables 4-7 through 4-9). The values calculated for this study are in relatively good agreement for the 30-year exposure periods. However at the 50-year and 100-year exposure period, the DWR values are generally higher for the Sherman Island site. The close proximity of this westernmost site to the faults may account for this.

Table 4-5: Annual Probabilities of Exceedance

PEAK BEDROCK ACCELERATION (g)	ANNUAL PROBABILITIES OF EXCEEDANCE* FOR SITES:					
	OLD RIVER CROSSING		SHERMAN ISLAND		TERMINOUS	
0.15	3.30E-02	1.18E-02	7.04E-02	2.87E-02	9.31E-03	3.16E-03
0.25	7.75E-03	2.11E-03	2.07E-02	7.06E-03	1.87E-03	4.86E-04
0.35	2.53E-03	5.36E-04	8.56E-03	2.52E-03	5.67E-04	1.14E-05

*Shaded figures are for magnitude weighing (M- 7.5) with respect to liquefaction.

Table 4-6: Return Periods of Peak Bedrock Accelerations

RETURN PERIOD (YEARS)	PEAK BEDROCK ACCELERATION* (g) FOR SITES:					
	OLD RIVER CROSSING		SHERMAN ISLAND		TERMINOUS	
50	0.180	0.124	0.254	0.172	0.115	0.081
100	0.230	0.158	0.330	0.221	0.146	0.105
1,000	0.448	0.302	0.704	0.460	0.300	0.207

*Shaded figures are for magnitude weighing (M = 7.5) with respect to liquefaction.

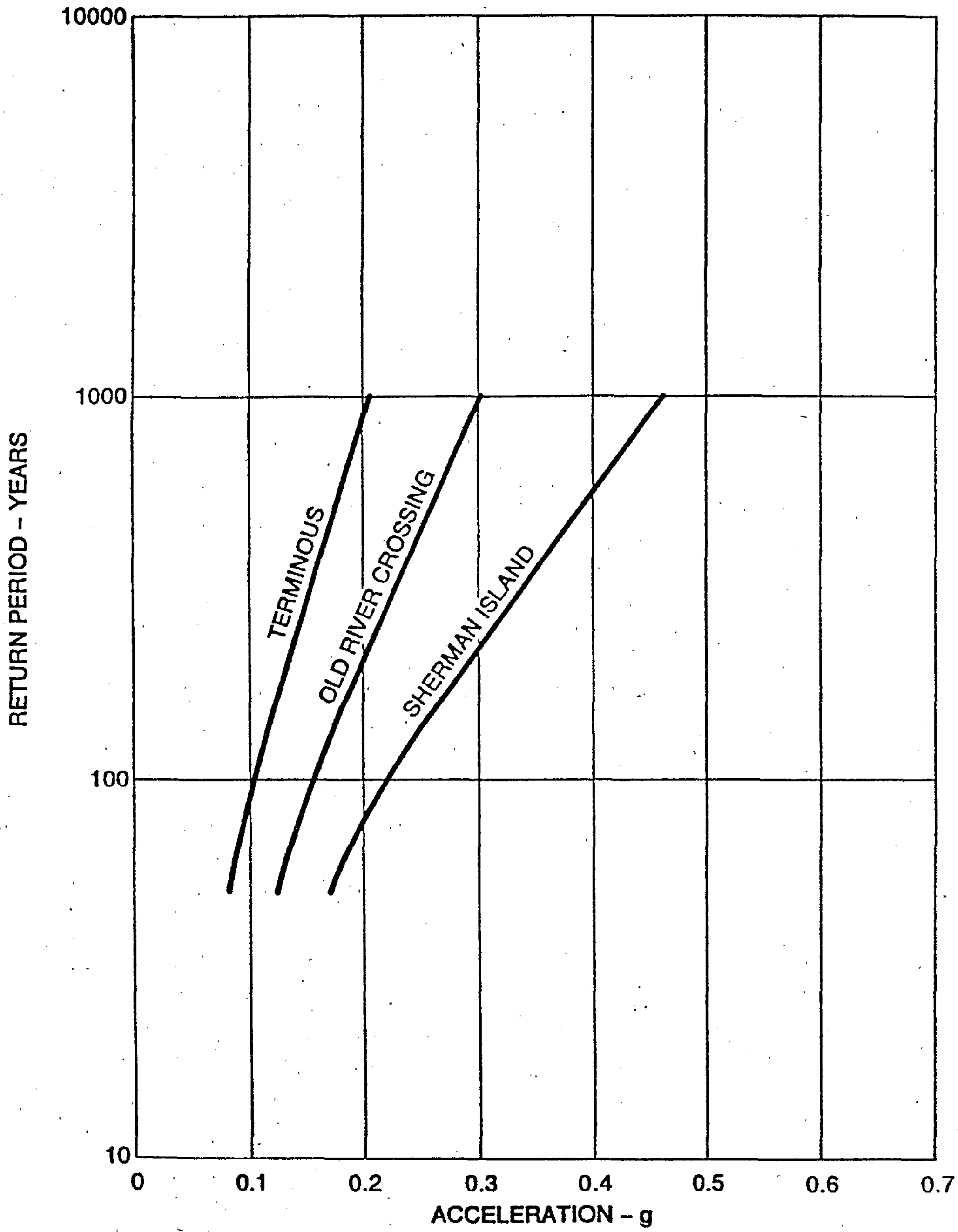


Figure 4-15: Return Period vs Peak Horizontal Acceleration For Three Delta Sites.

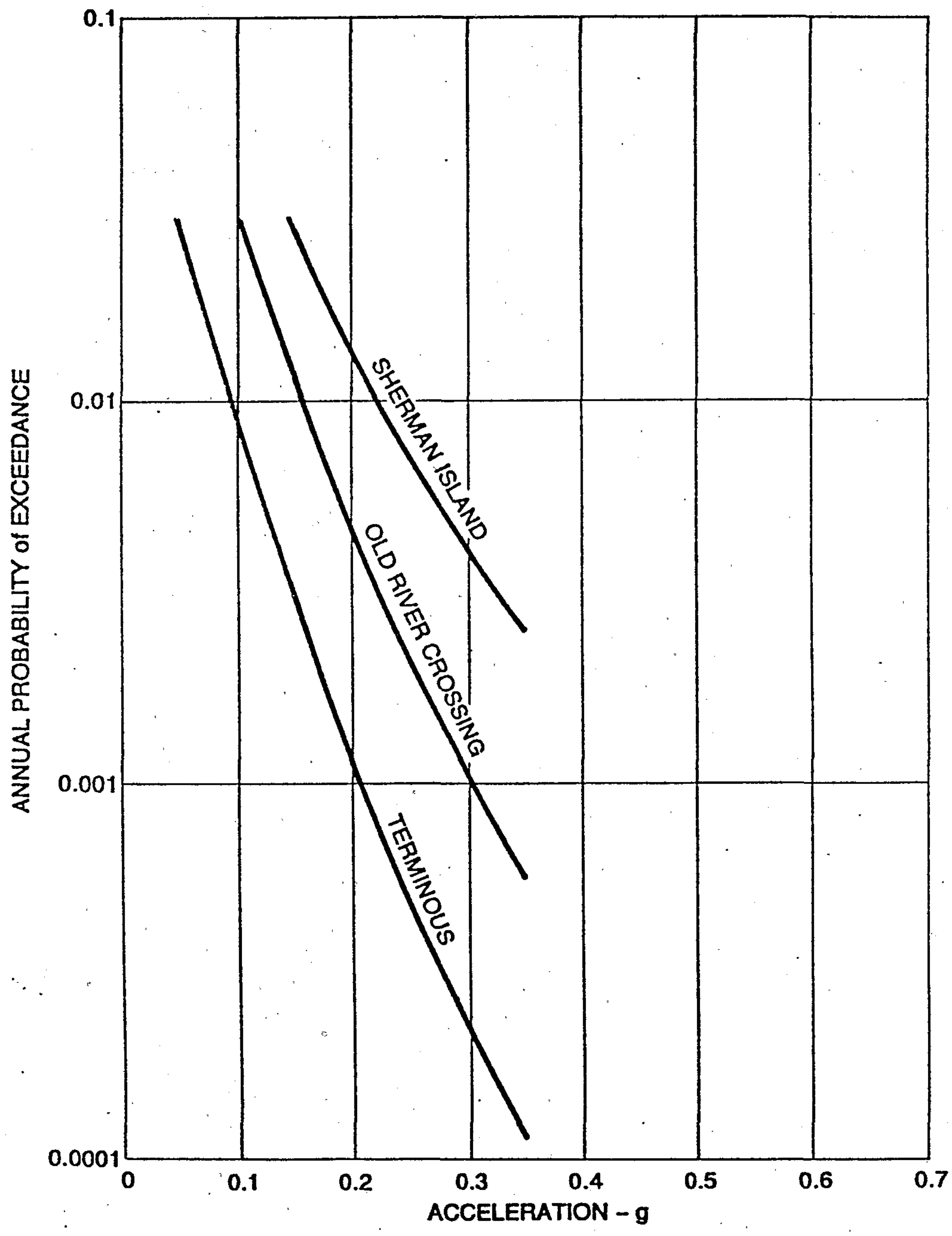


Figure 4-16: Annual Probability of Exceedance vs Acceleration for Three Delta Sites

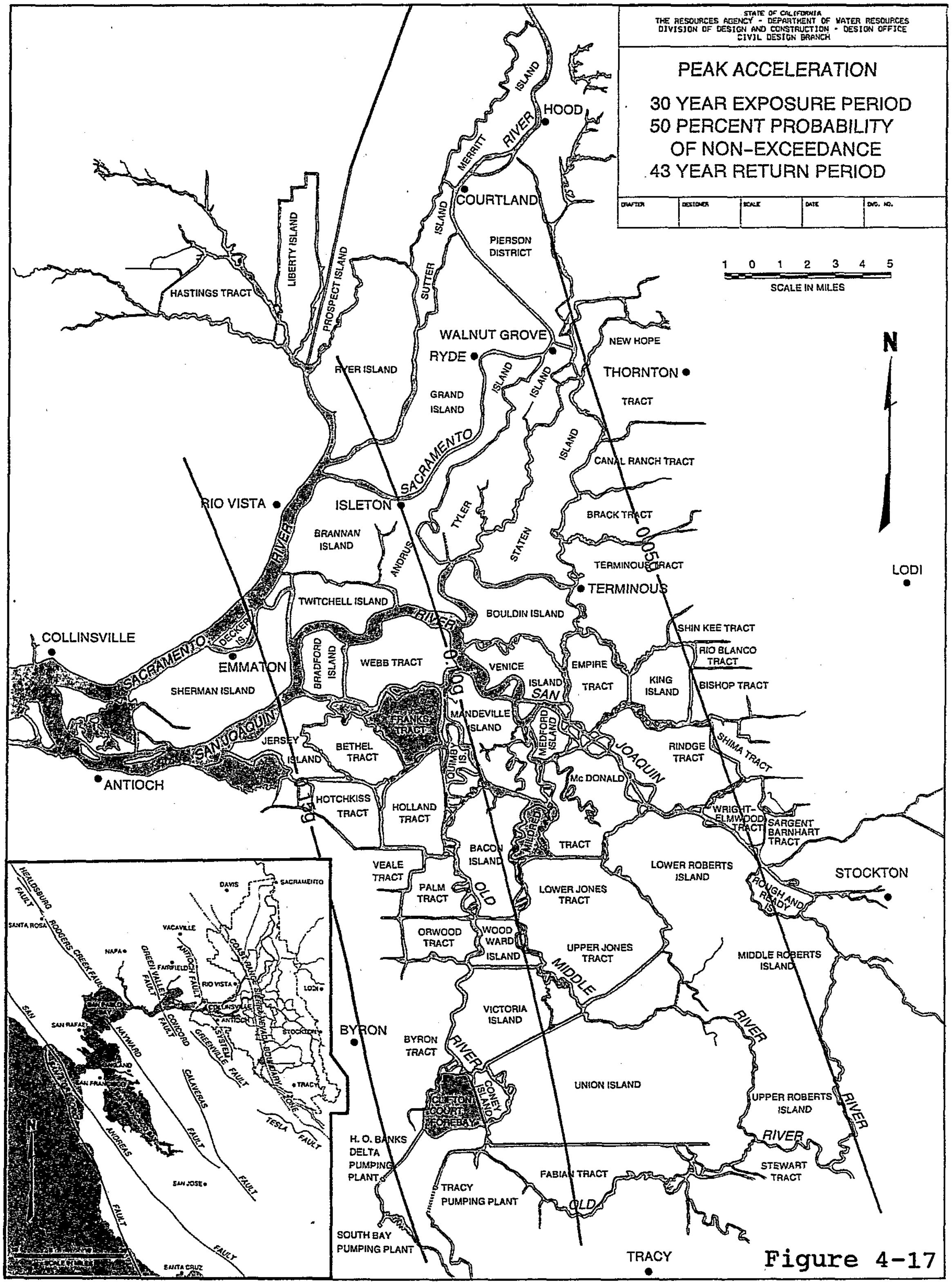
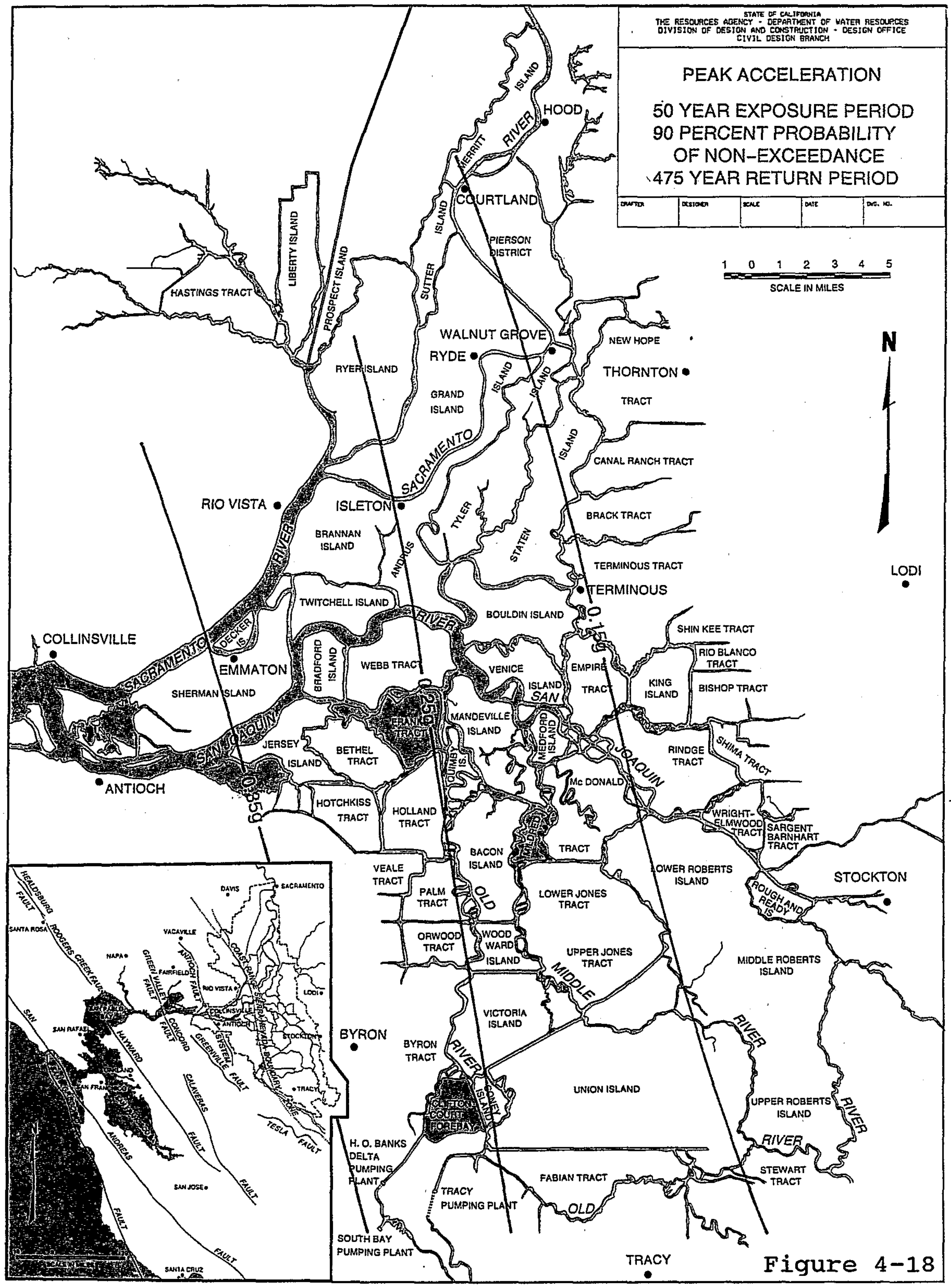

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
PEAK ACCELERATION
30 YEAR EXPOSURE PERIOD
50 PERCENT PROBABILITY
OF NON-EXCEEDANCE
43 YEAR RETURN PERIOD
SCALE IN MILES
N
HOOD
COURTLAND
PIERSON DISTRICT
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
SUTTER ISLAND
MERRITT ISLAND
WALNUT GROVE
RYDE
NEW HOPE
THORNTON
RYER ISLAND
GRAND ISLAND
CANAL RANCH TRACT
RIO VISTA
ISLETON
BRACK TRACT
BRANNAN ISLAND
TERMINOUS TRACT
TERMINOUS
LODI
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
COLLINSVILLE
EMMATON
BRADFORD ISLAND
WEBB TRACT
VENICE ISLAND
EMPIRE TRACT
KING ISLAND
BISHOP TRACT
SHERMAN ISLAND
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
RINDGE TRACT
SHIMA TRACT
JERSEY ISLAND
BETHEL TRACT
ANTIOCH
Mc DONALD
HOTCHKISS TRACT
HOLLAND TRACT
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
BACON ISLAND
LOWER ROBERTS ISLAND
STOCKTON
VEALE TRACT
PALM TRACT
LOWER JONES TRACT
ROUGH AND READY IS.
ORWOOD TRACT
WOODWARD ISLAND
UPPER JONES TRACT
MIDDLE ROBERTS ISLAND
VICTORIA ISLAND
BYRON
BYRON TRACT
UNION ISLAND
CLIFTON COURT FOREBAY
CONEY ISLAND
UPPER ROBERTS ISLAND
H. O. BANKS DELTA PUMPING PLANT
FABIAN TRACT
STEWART TRACT
TRACY PUMPING PLANT
SOUTH BAY PUMPING PLANT
TRACY
SACRAMENTO RIVER
SAN JOAQUIN RIVER
OLD RIVER
MIDDLE RIVER
0.05
0.10
DAVIS
SACRAMENTO
SANTA ROSA
VACAVILLE
NAPA
FAIRFIELD
RIO VISTA
COLLINSVILLE
ANTIOCH
LODI
STOCKTON
SAN RAFAEL
TRACY
SAN JOSE
SANTA CRUZ
HEALDSBURG FAULT
ROGERS CREEK FAULT
GREEN VALLEY FAULT
ANTIOCH FAULT
CONCORD FAULT
HAYWARD
CALAVERAS FAULT
SAN ANDREAS FAULT
TESLA FAULT
GREENVILLE FAULT

Figure 4-17


STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
PEAK ACCELERATION
50 YEAR EXPOSURE PERIOD
90 PERCENT PROBABILITY
OF NON-EXCEEDANCE
475 YEAR RETURN PERIOD
SCALE IN MILES
N
0.15g
0.25g
0.35g
HOOD
COURTLAND
WALNUT GROVE
RYDE
NEW HOPE
THORNTON
RIO VISTA
ISLETON
TERMINOUS
LODI
COLLINSVILLE
EMMATON
ANTIOCH
STOCKTON
BYRON
TRACY
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
RYER ISLAND
GRAND ISLAND
SUTTER ISLAND
MERRITT ISLAND
PIERSON DISTRICT
BRANNAN ISLAND
TWITCHELL ISLAND
BOULDIN ISLAND
SHERMAN ISLAND
BRADFORD ISLAND
WEBB TRACT
VENICE ISLAND
JERSEY ISLAND
BETHEL TRACT
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
QUIMBY IS.
HOTCHKISS TRACT
HOLLAND TRACT
BACON ISLAND
Mc DONALD TRACT
VEALE TRACT
PALM TRACT
ORWOOD TRACT
WOODWARD ISLAND
LOWER JONES TRACT
UPPER JONES TRACT
VICTORIA ISLAND
BYRON TRACT
UNION ISLAND
CLIFTON COURT FOREBAY
H. O. BANKS DELTA PUMPING PLANT
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SOUTH BAY PUMPING PLANT
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KING ISLAND
EMPIRE TRACT
RINDGE TRACT
SHIMA TRACT
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
LOWER ROBERTS ISLAND
ROUGH AND READY IS.
MIDDLE ROBERTS ISLAND
UPPER ROBERTS ISLAND
STEWART TRACT
SACRAMENTO RIVER
SAN JOAQUIN RIVER
OLD RIVER
MIDDLE RIVER
TYLER ISLAND
STATEN ISLAND
ANDRUS ISLAND
DECKER IS.
SAN ANDREAS FAULT
HAYWARD FAULT
CALAVERAS FAULT
GREENVILLE FAULT
CONCORD FAULT
ANTIOCH FAULT
TESLA FAULT
SACRAMENTO
DAVIS
VACAVILLE
NAPA
FAIRFIELD
SANTA ROSA
SAN RAFAEL
SAN JOSE
SANTA CRUZ
HEALDSBURG FAULT

Figure 4-18

Table 4-7: Peak Bedrock Acceleration for 30 Year Exposure, 50 Percent Probabilty of Non-Exceedance

	SHERMAN ISLAND	OLD RIVER AT MOKELUMNE AQUEDUCT	TERMINOUS
DWR (Current Study)	0.25 (0.17)	0.17 (0.12)	0.11 (0.07)
Dames & Moore (1991)	---	---	---
Converse Ward Davis Dixon (1982)	---	0.18	---
Harding Lawson and Associates (1991)	---	---	0.10
U. S. Bureau of Reclamation (1991)	---	---	---
Earth Sciences Association (1991)	---	0.14	---

*Note: Values in parenthesis represent normalization of the attenuation relationship for M = 7.5 with respect to liquefaction.

Table 4-8: Peak Bedrock Acceleration for 50 Year Exposure, 50 Percent Probability of Non-Exceedance

	SHERMAN ISLAND	OLD RIVER AT MOKELUMNE AQUEDUCT	TERMINOUS
DWR (Current Study)	0.29 (0.20)	0.21 (0.14)	0.13 (0.09)
Dames & Moore (1991)	0.16	0.15	0.12
Converse Ward Davis Dixon (1982)	---	0.21	---
Harding Lawson and Associates (1991)	---	---	0.13
U. S. Bureau of Reclamation (1991)	---	---	---

*Note: Values in parenthesis represent normalization of the attenuation relationship for M = 7.5 with respect to liquefaction.

Table 4-9: Peak Bedrock Acceleration for 100 Year Exposure
90 Percent Probability of Non-Exceedance

	SHERMAN ISLAND	OLD RIVER AT MOKELUMNE AQUEDUCT	TERMINOUS
DWR (Current Study)	0.68 (0.45)	0.44 (0.30)	0.29 (0.21)
Dames & Moore (1991)	0.27	0.25	0.23
Converse Ward Davis Dixon (1982)	---	0.27	---
Harding Lawson and Associates (1991)	---	---	0.36
U. S. Bureau of Reclamation (1991)	0.45	---	0.22

*Note: Values in parenthesis represent normalization of the attenuation relationship for M = 7.5 with respect to liquefaction.

5. LEVEE DAMAGE CAUSED BY PAST EARTHQUAKES

5.0 SUMMARY

A review of available historical information indicates that there has been little damage in the Delta caused by historical earthquakes. No report could be found to indicate that an island or tract had been flooded due to an earthquake-induced levee failure. Further, no report could be found to indicate that significant damage had been induced by earthquake shaking. The minor damage that has been reported has not significantly jeopardized the stability of the Delta levee system.

This lack of severe earthquake-induced levee damage corresponds to the fact that no significant earthquake motion has apparently ever been sustained in the Delta area since the construction of the levee system approximately a century ago. Consequently, the lack of damage should not lead, necessarily, to a conclusion that the levee system is not vulnerable to moderate to strong earthquake shaking. The levee system simply has never been significantly tested. The particular findings from this historical review are as follows:

- Damage intensity maps indicate that, since the reclamation of the Delta islands began in the late 1860s, stiff soils and rock profiles at the periphery of the Delta have experienced peak accelerations no higher than about 0.1g to 0.15g. Within the central portions of the Delta, outcrops of stiff soil or rock would have experienced a peak acceleration of no more than about 0.10g.

- The most severe reported damage to levees in the Delta due to earthquake shaking appears to be the approximate 3 feet of settlement reported at a Santa Fe Railroad Bridge across Middle River following the 1906 San Francisco Earthquake. However, no flooding resulted.

- The damage to Delta levees reported by Finch (1985) as being caused by earthquakes occurring between 1979 and 1984 is in most cases difficult to definitively attribute to earthquake shaking. According to reinterviews of witnesses, there often was pre-earthquake distress at most of the sites mentioned in the Finch report. In addition, some of the damage reported may be related to other factors (e.g., ongoing levee subsidence or levee modifications being made at the time of the earthquake). Further, even those incidents which were verified do not indicate a level of damage significantly above that which existed prior to the earthquake. Consequently, these incidents do not provide additional insight on the relative vulnerability of Delta levees to seismic shaking.

- There was significant damage to a Southern Pacific railroad embankment on soft soil west of the Delta in the Suisun Marsh during the 1906 San Francisco Earthquake. The embankment settled several feet for a significant length (some reports indicate as much as 3 to 6 feet of settlement for over a thousand feet). This location had previously experienced distress the previous year and the earthquake-induced damage appears to be related to bearing failure rather than liquefaction. An outcrop of rock or stiff soil at this location would have been expected to have sustained peak ground accelerations of about 0.18g during this Magnitude 8+ event on the San Andreas Fault.

5.1 HISTORICAL REVIEW

A historical review of past earthquakes and earthquake-induced damage in the Sacramento-San Joaquin Delta was performed in order to develop a better understanding of the susceptibility of Delta levees to damage during future earthquakes. To this end, accounts of earthquake shaking and damage were collected and reviewed.

One particular report published by Michael Finch in the February 1985 issue of California Geology and entitled "Earthquake Damage in the Sacramento-San Joaquin Delta," reports levee damage occurring between 1979 and 1984 from relatively remote and/or small earthquakes. The case-histories reported by Finch (1985) are described in subsequent sections. As this report is the only known published account of such damage in recent years, the case histories of damage were subject to particular research during this review.

5.11 Sources of Information

The historical review was concentrated in the six-county area which encompasses the Sacramento-San Joaquin Delta (see Figure 5-1). The investigation employed several information sources:

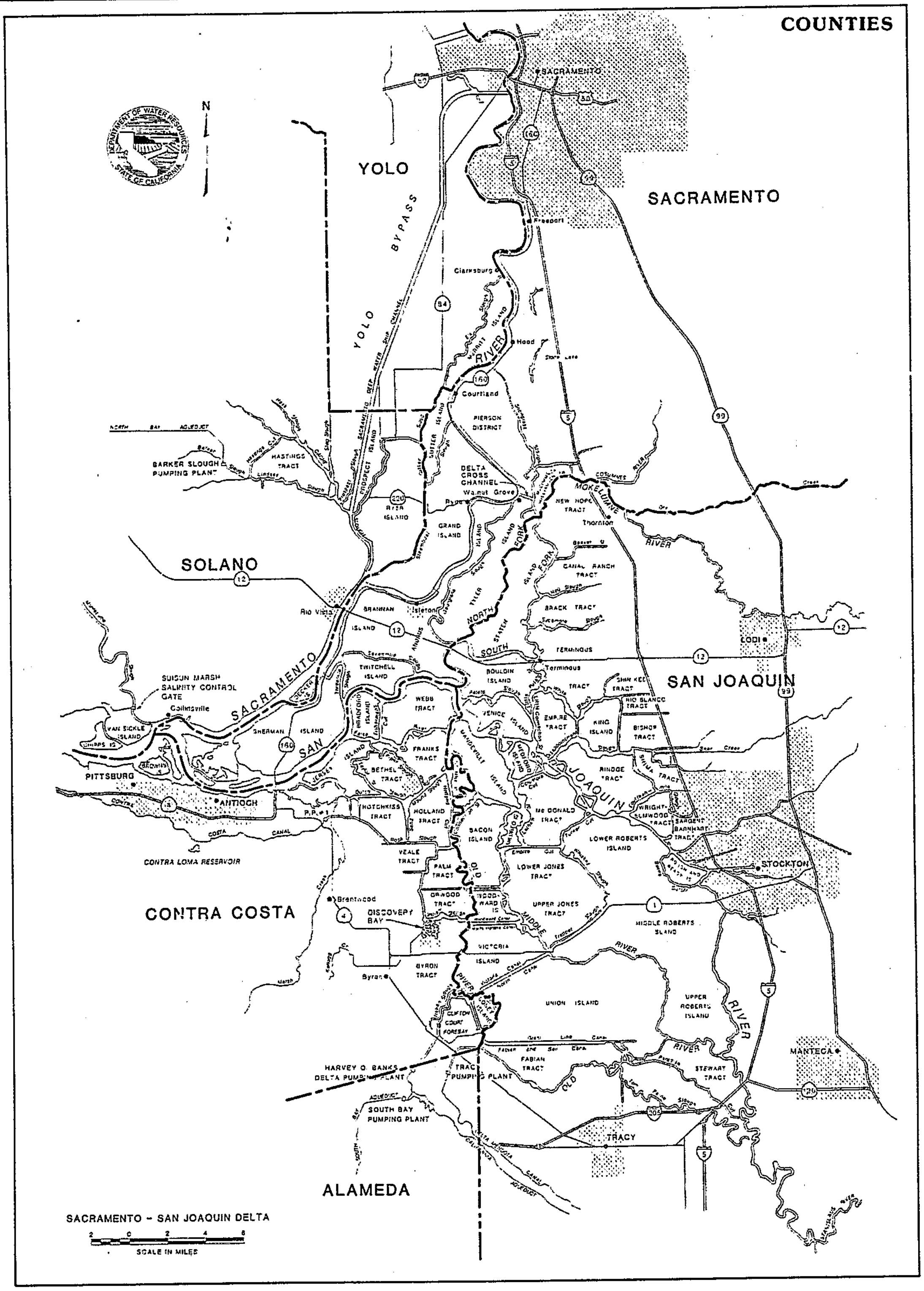


Figure 5-1: Six-County Area Encompassing the Delta
(From DWR, 1987)

- Review of engineering reports, newspaper articles, libraries, city and state archives, museums, and utility damage reports.

- Personal interviews with Delta residents, levee inspectors, maintenance personnel, district engineers, and State and Federal representatives.

- Personal inspections of Delta islands and levees to view locations of reported damage.

Appendices C through E contains copies of documents collected and reviewed for this investigation.

5.12 Limitations of Damage Assessments

Early in the investigative process, it became apparent that the available information was meager and often incomplete or contradictory. Consequently, when possible, published reports of recent damage were reinvestigated. This sometimes produced valuable additional information. Factors which complicated the evaluation of reported damage included the following:

Remote and Unpopulated Region - The island tracts in the Delta are relatively removed from populated areas and the major portions of the levees are generally not frequented by the public. As Youd (1978) observed: "areas in and near centers of population, along major transportation routes, and along major faults have generally received much more attention than less developed, less relevant or more remote areas."

Background Levee Distress - Delta levees are marginal earth structures with respect to stability under normal conditions. On any given day, a trip to various Delta islands will show levee reaches which have numerous cracks and seepage areas. This background level of distress is common to many island levees and is not terribly different than the types and levels of damage generally attributed to earthquake shaking.

Lack of Documented Inspections - Although the levees on each island are generally inspected on a routine basis by reclamation district staff, inspections are generally more focused and intensified immediately after an earthquake. Consequently, there is a tendency to discover damage following an earthquake that may have already existed prior to the event. Because there is often a lack of detailed and/or documented before-earthquake information, it is generally quite difficult to determine the amount of damage that is exclusively earthquake related.

Potential Motivations Behind Damage Reports - Previous investigators have stated that political and economic factors may influence damage reports. These factors are generally related to availability of government funds to rebuild damaged or upgrade pre-existing facilities and/or fears relating to devaluation of property following earthquake damage. Examples where previous investigators have made these charges are as follows:

- Youd and Hoose (1978) state that post-earthquake damage assessments concerning the 1868 Great Hayward Earthquake appear to have been intentionally suppressed.
- Hansen and Condon (1989) allege that damage reports following the 1906 San Francisco Earthquake were reduced and/or suppressed for fear of property devaluation and loss of capital investment.
- The Federal Emergency Management Agency criticized the timing of code changes made in the Port of Oakland and the City and County of San Francisco, alleging they were made to get Federal relief for damage caused by the Loma Prieta earthquake (Engineering News Record, October 1991).

5.2 HISTORICAL EARTHQUAKES AND ESTIMATES OF PEAK GROUND ACCELERATIONS

5.21 Historical Earthquakes

The earliest reported earthquake to occur near the Delta was the June 21, 1808 event (USGS, 1990). This was a moderate earthquake occurring in the San Francisco region. This earthquake event was estimated to be a Magnitude 6 event, based on the effects of the earthquake on the region according to damage reports (Toppozada, 1981). On June 10, 1836, a moderate earthquake, M = 6.8, occurred on the Hayward fault. And in June 1838, an M = 7 event occurred on the San Francisco peninsula. Very little is known about the effects of these earthquakes on the Delta areas. This is because the Delta was essentially unreclaimed during this time and the Delta region was populated chiefly by Native Americans and there were no significant European or American settlements nearby (DWR, 1982).

A review of studies performed by Toppozada (1981) and others, indicate that between 1855 and 1989, approximately 55 earthquakes with magnitudes above 4.5 occurred close enough to the Delta to inflict observable damage. These earthquakes are listed in Table 5-1 along with the range in Modified Mercalli Intensity induced in the Delta region by the earthquake.

5.22 Estimates of Peak Ground Acceleration

Peak ground accelerations were estimated by correlating Modified Mercalli Intensities for earthquakes that affected the Delta between 1855 and 1989. Relationships developed by Trifunac and Brady (1975) between Modified Mercalli Intensity values and peak horizontal accelerations were used to establish the following correlations:

Modified Mercalli Intensity	Peak Acceleration (g)
IV	0.015 - 0.02
V	0.02 - 0.05
VI	0.05 - 0.08
VII	0.08 - 0.18
VIII	0.18 - 0.32

Figure 5-2 shows a typical isoseismal contour map developed by Toppozada (1981) for one of the historical earthquakes. In this case, the event is the October 21, 1868 earthquake (M = 6.8) and the figure shows that the Modified Mercalli Intensity for most of the Delta would be between values of VI and VII. For each of the 55 earthquakes between 1855 and 1989, peak ground accelerations were determined for three Delta sites: Sherman Island, Terminous Tract, and Clifton Court Forebay. Table 5-2 tabulates the peak ground accelerations estimated using this method. Appendix B contains all of the intensity maps used for the acceleration determination.

It should be noted that Trifunac and Brady say that the basis for correlating an earthquake intensity scale with the recorded levels of strong ground motion is dubious and that one must understand that the accuracy is poor. There is wide scatter in the data, and there is sometimes a lack of a physical basis for the correlation.

Table 5-1
Estimated Peak Ground Accelerations Based on Modified Mercalli Intensities for the Sacramento-San Joaquin Delta Region 1850 - 1989

Earthquake Date	Modified Mercalli Intensity	Estimated Magnitude	Peak Ground Accelerations (g) Sherman Island	Terminous Tract	Clifton Court
August 27, 1855	IV	4.9	0.01	0.01	0.01
January 2, 1856	IV	5.3	0.01	0.01	0.01
February 15, 1856	V	5.5	0.02	0.01	0.01
November 26, 1858	VI	6.1	0.06	0.03	0.07
July 4, 1861	V	5.6	0.04	0.02	0.05
December 19, 1863	II-IV	4.8	0.01	0.01	0.01
February 6, 1864	V	5.9	0.02	0.01	0.01
March 5, 1864	IV-V	5.7	0.04	0.04	0.03
May 21, 1864	II-V	5.3	0.01	0.01	0.02
July 22, 1864	II-IV	4.7	NF	NF	NF
March 8, 1865	II-IV	4.7	NF	NF	NF
October 8, 1865	V-VI	6.3	0.06	0.04	0.07
March 26, 1866	II-IV	5.4	NF	NF	0.01
July 15, 1866	V	5.8	0.02	0.02	0.05
October 21, 1868	VI-VII	6.8	0.08	0.05	0.08
February 17, 1870	II-IV	5.8	NF	NF	0.02
April 2, 1870	II-IV	5.3	0.01	NF	0.02
April 10, 1881	V-VI	5.9	0.04	0.02	0.05
March 6, 1882	II-IV	5.7	NF	NF	NF
March 30, 1883	II-V	5.6	NF	NF	NF
March 26, 1884	II-V	5.9	NF	NF	NF
March 31, 1885	II-IV	5.5	NF	NF	NF
April 2, 1885	II-IV	5.4	NF	NF	NF
April 12, 1885	II-V	6.2	NF	NF	0.01
April 15, 1889	II-IV	4.8	NF	NF	NF
May 19, 1889	VI-VII	6.0	0.13	0.05	0.06
July 31, 1889	II-IV	5.2	0.01	NF	0.01
April 24, 1890	V	6.0	0.03	0.02	0.05
January 2, 1891	II-IV	5.5	NF	NF	0.01
October 12, 1891	V	5.5	0.03	0.02	0.03
April 19, 1892	VI	6.4	0.06	0.05	0.06
April 21, 1892	V	6.2	0.04	0.05	0.02
April 30, 1892	V	5.5	0.04	0.04	0.02
November 13, 1892	V	5.6	0.02	0.01	0.03
June 30, 1893	II-IV	4.6	NF	NF	NF
August 9, 1893	II-IV	5.1	NF	NF	NF
June 20, 1897	V	6.2	0.03	0.01	0.05
March 31, 1898	V	6.2	0.05	0.03	0.05
April 30, 1899	II-IV	5.6	NF	NF	0.01
June 2, 1899	IV-V	5.4	0.01	NF	0.02
July 6, 1899	V	5.8	0.02	NF	0.04
May 19, 1902	V	5.4	0.04	0.03	0.03
June 11, 1903	V	5.8	0.04	0.03	0.05
August 3, 1903	V	5.8	0.04	0.03	0.05
April 18, 1906	VI-VII	8.3	0.08	0.06	0.08
July 1, 1911	VI	6.6	0.05	0.04	0.08
October 22, 1926	V	6.1	0.05	0.03	0.05
October 24, 1955	V	5.4	0.05	0.02	0.02
March 22, 1957	II-III	5.3	NF	NF	NF
August 6, 1979	II	5.8	<0.02	<0.02	<0.02
January 24, 1980	IV-VII	5.5	0.03	0.01	0.14*
January 26, 1980	VI-VII	5.8	0.08	0.06	0.07*
May 2, 1983	II-IV	6.7	0.02	0.02	0.02
April 24, 1984	II-IV	6.2	0.02	0.02	0.02
October 17, 1989	V	7.1	0.04	0.02	0.08**

Notes on following page.

NOTES:

1. Damage Intensities and Estimated Magnitudes between 1855 and 1982 are from Toppozada (1981, 1982). Values after 1982 obtained from USGS and CDMG determinations (see Appendix D).
2. NF denotes "not felt."
3. * Peak ground acceleration measured at Delta Pumping Plant (rock).
4. ** Peak ground acceleration measured at Clifton Court Forebay (soil).

The estimated peak ground accelerations shown in Table 5-1 for previous earthquakes indicate that the Delta has never experienced even moderate levels of earthquake accelerations. The largest peak acceleration shown is the 0.14g value at Clifton Court Forebay, which was actually measured at the nearby Delta Pumping Plant during the January 24, 1980 Livermore Earthquake. All of the other estimated peak ground accelerations are equal to 0.1g or less. Even the 1906 San Francisco Earthquake is estimated to have generated peak ground accelerations of 0.08g or less within most of the Delta region.

5.3 DAMAGE REPORTS FROM SACRAMENTO-SAN JOAQUIN DELTA

As mentioned in a previous section, the earliest known earthquake near the Sacramento-San Joaquin Delta occurred on June 21, 1808. However, since reclamation of the Delta did not begin until the late 1860s and because the area was largely unpopulated, no reports of damage were apparently made for this event. This result was common in the Delta for many of the earthquakes which occurred during the 19th century. To facilitate the review of historical damage, earthquakes occurring within the following periods of time were grouped together:

- 1850 to 1891
- 1892 to 1905
- The 1906 San Francisco Earthquake
- 1907 to 1978
- 1979 to 1988
- The 1989 Loma Prieta Earthquake

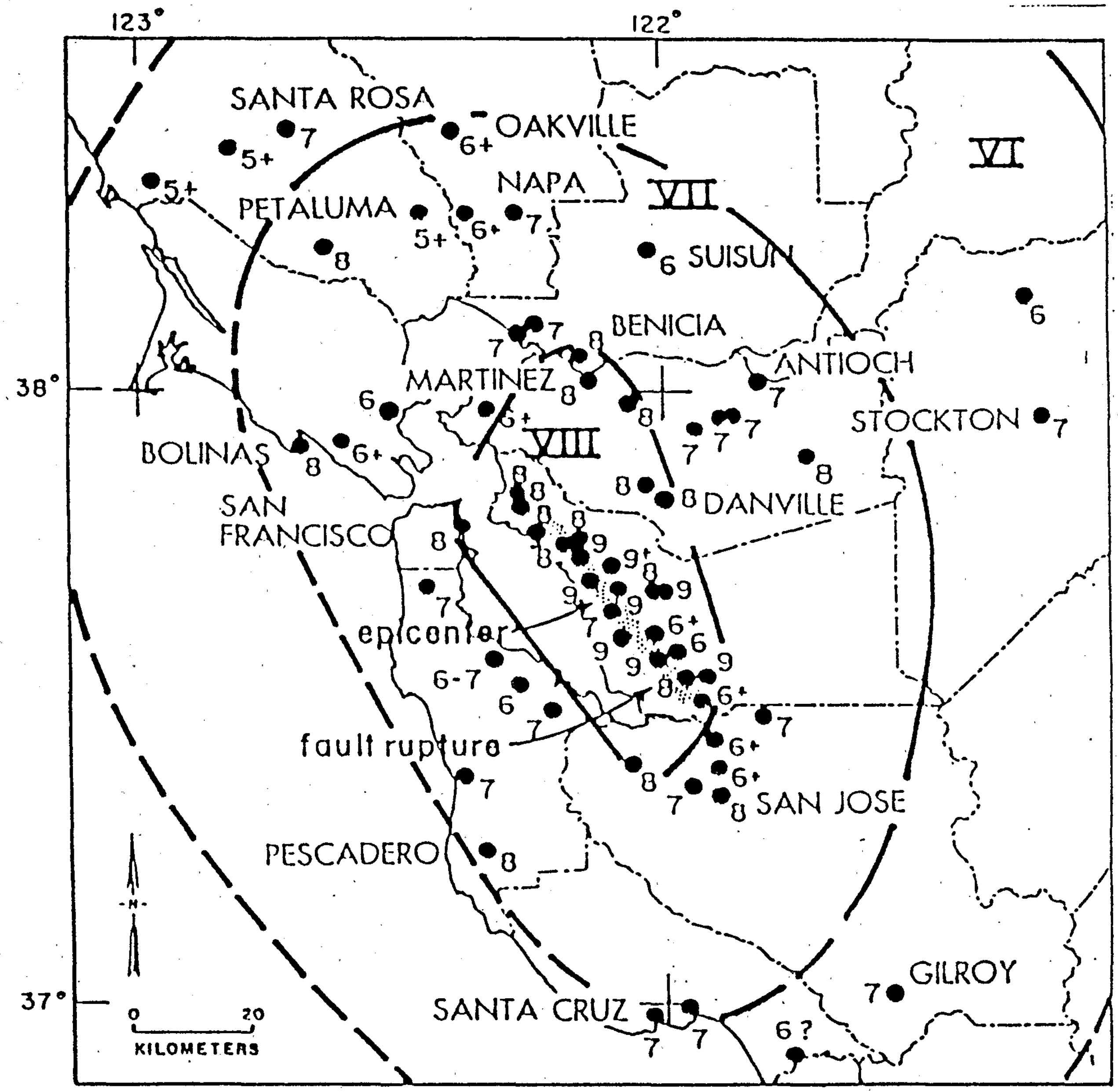


Figure 5-2: Isoseismal Map of Modified Mercalli Intensities From the October 21, 1868 Hayward Earthquake (From Toppozada, 1981)

5.4 DAMAGE REPORTS FROM EARTHQUAKES BETWEEN 1850 AND 1891

As detailed in previous sections and in Table 5-1, there were approximately 30 earthquakes between 1855 and 1891 with high enough magnitudes and close enough proximities to possibly induce observable damage in the Delta. However, literature searches were unable to discover any reports of damage within the Delta region during this time period.

5.5 DAMAGE REPORTS FROM EARTHQUAKES BETWEEN 1892 AND 1905

Table 5-1 shows that there were 14 earthquakes between 1892 and 1905 with high enough magnitudes and close enough proximity to possibly induce observable damage in the Delta. The most prominent events were the April 19 and 21, 1892 Vacaville-Winters earthquakes with estimated magnitudes of 6.4 and 6.2. These events induced damage in the Winters and Vacaville communities, as well as slumping along Putah Creek near Winters. Within the Delta region, however, the Modified Mercalli Intensities for these events were only at levels V and VI, and no report of significant damage within the Delta could be found. The lack of reported damage was also true for the other earthquakes which occurred within this time period.

The two 1892 earthquakes are of interest because their estimated locations are along the western margin of the Central Valley. Recent studies by Wong (1988) and the USGS (1991) suggest that these earthquakes may have developed on a low angle, blind thrust fault that might be similar in nature to the source of the 1983 Coalinga earthquake, located on the western margin of the Central Valley further south. There has been additional speculation that there might be a series of en echelon blind thrust faults along the Coast Range-Sierra Nevada Boundary Zone. Such a fault could theoretically run through the middle of the Delta and be the source of the largest potential earthquake shaking for most of the Delta region (see Chapter 2).

5.6 DAMAGE REPORTS FROM THE 1906 SAN FRANCISCO EARTHQUAKE

The earthquake of April 18, 1906 (Estimated Richter Magnitude 8.3), is one of the most significant earthquakes of recent time. The northernmost 270 miles of the San Andreas Fault from San Juan Bautista to Cape Mendocino ruptured and strong shaking lasted from 45 to 60 seconds. Lawson (1908) reports that the earthquake was felt from Southern Oregon to south of Los Angeles and inland as far as central Nevada (see Figure 5-3).

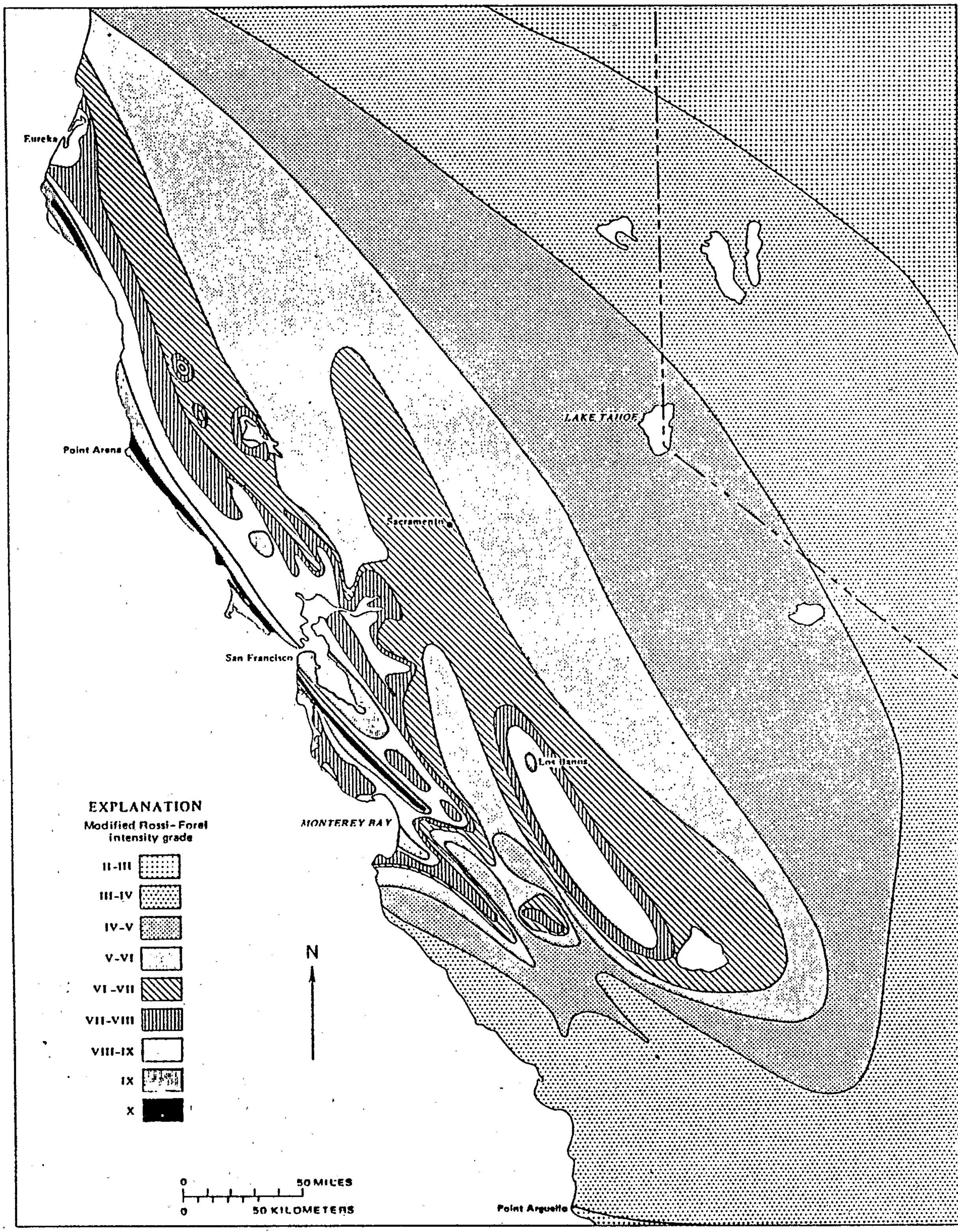


Figure 5-3: Isoseismal Map of Modified Rossi Forel Damage Intensities for the 1906 San Francisco Earthquake (From Lawson, 1908)

5.61 Pre-Earthquake Conditions in the Delta

By 1906, much of the current Delta had been reclaimed by the establishment of levee systems and inland pumping. Figure 5-4 presents a map of the Sacramento-San Joaquin Delta as it was in 1901. Presumably, the channels and reclaimed islands shown were not very different than those during the 1906 earthquake. However, many of the islands and levee systems that exist today were not in existence at this time (e.g., levees systems along more recent manmade channels such as Bishop's, Empire, Fisherman's, Holland, and Honker Cuts). More importantly, the levee heights in 1906 were significantly smaller than those which commonly exist in the Delta today. On most islands today, levee heights are typically between 15 and 25 feet. However, at the time of the 1906 earthquake, the levee heights were probably between 5 and 15 feet. Thus, the levees in 1906 were somewhat smaller in number and about half the size of those which exist today.

During the time of the 1906 earthquake, it was common for Delta islands to flood from the results of winter storms and/or Spring runoff. A few months prior to the earthquake, heavy precipitation was being reported throughout northern California, causing flooding in or near the Delta Region. On January 11, 1906, the Lodi Sentinel reported seven inches of rain in a 24-hour period. On March 17th, flooding was being reported on the eastern edges of the Delta at Woodbridge, Bear Creek, and Dry Creek. On March 27th, a levee on the Mokelumne River broke inundating over one thousand acres to the east of the Delta. In addition, heavy rain continued after the earthquake causing flooding in the region well into the summer. Eleven Delta islands or tracts were inundated by floodflows within three months after the earthquake. Consequently, the occurrence of the heavy rain at this time obscures potential damage which might have occurred only from the earthquake.

5.62 Damage Reports for Delta Sites from The 1906 Earthquake

Following the 1906 Earthquake, four cases of ground failure or distress were reported for sites in the Sacramento-San Joaquin Delta. These sites are as follows:

1. Near Woodbridge (near Lodi), the bed of the Mokelumne River dropped 12 feet and Tracy Lake almost drained (San Francisco Chronicle, May 1, 1906).

2. On Bouldin Island the levee opened at the location of an old seep, a temporary increase in flow developed, and then the opening closed and all seepage stopped (Lodi Sentinel, May 5, 1906).

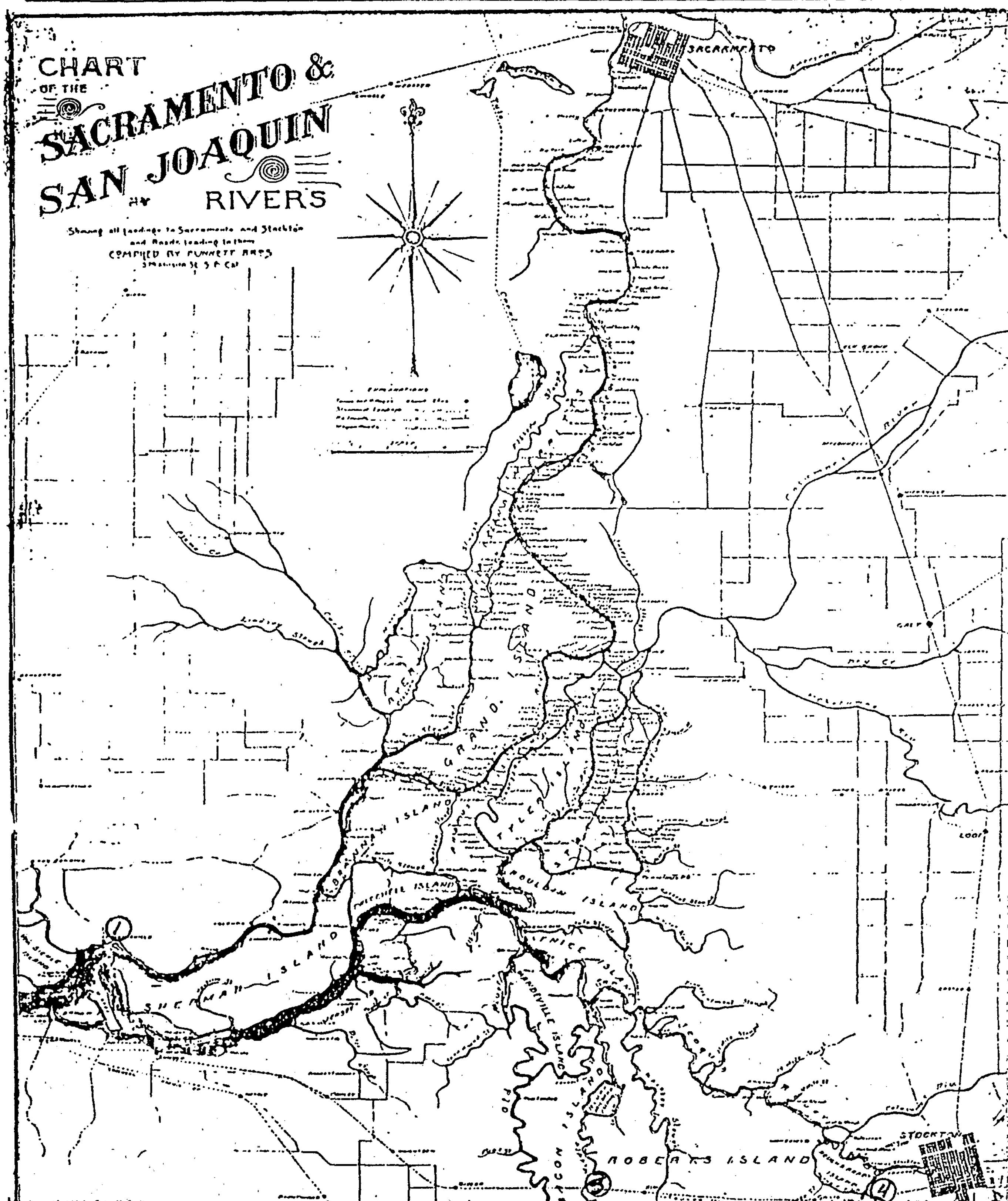


Figure 5-4: 1901 Map of the Sacramento-San Joaquin Delta

3. A railway bridge over the San Joaquin River near Stockton settled several inches (New York Tribune, April 19, 1906).

4. The Santa Fe Railroad bridge at Middle River between Point Richmond and Stockton sank three feet and was twisted out of line (The Salinas Daily Index, April 20, 1906). It is not clear if the approach fills settled or the bridge foundation settled.

In general, the reported damage is relatively minor considering the severity of the earthquake in the San Francisco metropolitan area. However, it is consistent with the generally low level damage intensity reported for the Delta region. According to Lawson (1908), the Rossi-Forel intensity for the Delta region was about VI to VII for all but the westernmost tip of Sherman Island, which was about VII to VIII (see Figure 5-3). This corresponds to a Modified Mercalli Intensity of about VI to VIII. According to the relationship developed by Trifunac and Brady (1975), the peak horizontal acceleration on rock or stiff soil outcrops near Sherman Island would be only about 0.08g, and only about 0.06g at Terminous.

The worst damage in the Delta from this event was the three feet of settlement reported for the Middle River railroad bridge. This damage is the most severe earthquake-induced distress ever reported for the Delta region.

5.63 Post-Earthquake Flooding of Delta Islands

Finch (1985) suggests that the April 18, 1906 San Francisco Earthquake may have weakened the overall Delta levee system, leading to widespread levee failures and island inundation during the following year in 1907. However, as mentioned previously, widespread flooding was common during this period of time. In 1904, floodflows were responsible for the inundation of 12 islands in the Delta. Furthermore, the levee system was tested by floodwaters within three months after the 1906 Earthquake, leading to levee failures and inundation of 11 islands. Newspaper accounts indicate that there was very little levee freeboard during these floodflows, suggesting that many of the failures were due to overtopping. According to the Lodi Sentinel and USACE (1982), the number of islands flooded are as follows:

Number of Islands Flooded		
1904	June-July 1906*	1907
12	11	21

* 2-3 months after earthquake

The 1906 high water, occurring two to three months after the earthquake, would seem to be a more time-relevant test than the 1907 period of high water. However, the 11 islands flooded in 1906 after the earthquake are not terribly different in number than the 12 islands that were flooded in 1904 prior to the earthquake. Consequently, the suggestion that the 1906 Earthquake was responsible for the inundation of 21 islands in 1907 does not appear to have very much support.

5.64 Damage Reports for Organic Soil Sites to the West of the Delta from the 1906 Earthquake

Areas to the west of the Delta presumably experienced higher ground motions because of their greater proximity to the fault rupture on the San Andreas Fault. Some of these areas contained facilities which were founded on organic or marshy soils and there were a few notable case histories of ground failure or distress associated with the 1906 Earthquake. The locations of these case histories are shown in Figure 5-5 and are as follows:

A. The railroad track east of Martinez, near Bull's Head Old Works, was thrown 3 inches out of alignment to the north. Many cracks occurred in the embankment on both sides of the track. A series of five small transverse waves were also found in the embankment about half a mile west of Peyton Station. The distance between crests was about 10 to 15 feet, with an approximate wave amplitude of 3 inches. This embankment lies in flat marshy land (Lawson, 1908).

B. The Southern Pacific Railroad embankment in the Suisun Marsh between the Sprig and Teal stations was reported to have sunk into the soft marshy ground for extensive reaches. According to the Evening Bee of April 18, 1906:

> "A short time after the big shock came a message from Suisun, Solano County, saying that a long section of track had disappeared from view. It was learned later that, in

> one place between Sprig and Teal Stations, in the Suisun Marshes, for a distance of one mile and a half, the track had sunk down 3 to 6 feet, and at another point nearly 1,000 feet of track went out... It was at the spot where the track disappeared that the railroad company had so much trouble last winter, when a loaded passenger train came near going out of site. A great army of men was then set at work to fill up the sink. The task was a most difficult one as enormous timbers which were thrown into the hole quickly sank from view and the trainloads of earth dumped in disappeared like snow in a fierce sunshine."

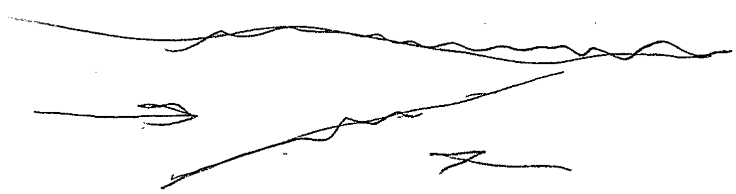
This account is suggestive of a bearing failure in soft organic soils rather than liquefaction of a sandy or silty embankment or foundation. Several other newspapers repeated this account and/or reported modified versions. Some accounts varied in the level of damage. However, a few days later Southern Pacific reported that the damage had been exaggerated and that only two or three carloads of dirt was required to level the tracks and the trains were running again.

c. In Collinsville, the Collinsville Hotel was completely destroyed. It was noted in the report that "Collinsville is on the peat of the tule land, with hard clay two feet below the surface" (Lawson, 1908).

Using conventional attenuation relationships (e.g., Idriss, 1985), outcrops of rock or stiff soil near the above three sites west of the Delta would have been expected to have peak ground accelerations between 0.15g and 0.18g of during the Magnitude 8+ event.

5.7 DAMAGE REPORTS FROM EARTHQUAKES BETWEEN 1906 AND 1978

The period between 1906 to about 1978 was a relatively quiet time for earthquakes in northwestern California. Table 5-1 shows only four earthquakes between 1906 and 1978 with high enough magnitudes and close enough proximities to possibly induce observable damage in the Delta. However, literature searches were unable to discover any reports of damage within the Delta region during this time period.



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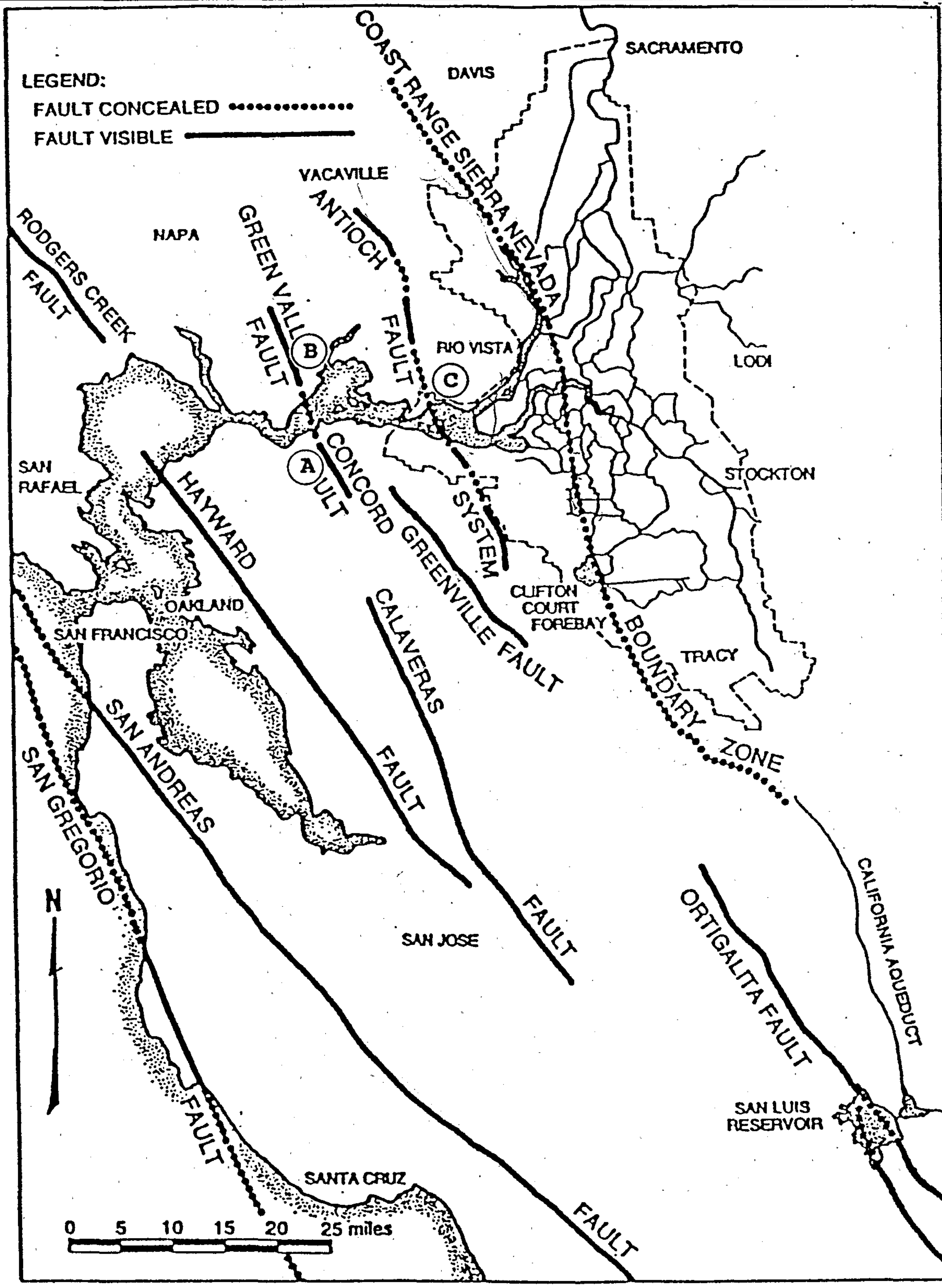


Figure 5-5: Damage Reports for Organic Soil Sites to the West of the Delta from the 1906 Earthquake

5.8 DAMAGE REPORTS FROM EARTHQUAKES BETWEEN 1979 AND 1989

Earthquake damage reported after 1979 allowed a more comprehensive investigation. In addition to newspaper accounts and independent reports of ground damage in the Delta Region, interviews with Delta residents, island caretakers, Reclamation District engineers, and others were conducted so as to better ascertain the nature of the reported damage.

5.81 Case Histories Reported by Finch (1985)

As mentioned in previous sections, several case histories of earthquake-induced ground damage occurring in the Delta between 1979 and 1984 were compiled in the February 1985 issue of California Geology in an article entitled, "Earthquake Damage in the Sacramento-San Joaquin Delta." This is essentially the only publication found that reports earthquake damage in the Delta during this time period and is widely quoted by other investigators. These case histories are based on reports from eyewitnesses who either worked or lived in the Delta, and in some cases were working at the particular site when the earthquake occurred. Table 5-2 summarizes these reports and Figure 5-6 shows the locations for the reported ground damage.

As indicated in Table 5-2, the case histories cited by Finch (1985) suggest that the Delta levees are extremely susceptible to damage from relatively distant earthquakes. Half of the case histories of levee damage are said to have occurred following the 1983 Coalinga Earthquake, located approximately 150 miles away. Another four case histories are reported for the 1979 Coyote and 1984 Morgan Hill Earthquakes, located approximately 60 miles away from the levees reported to be damaged. However, most of the reported earthquake-induced distress consists of minor cracking, a general condition of many Delta levees regardless of earthquake shaking. Consequently, it is difficult to definitively conclude that the earthquakes were responsible. To attempt to better evaluate these damage reports, interviews were conducted with the persons who gave the original accounts and with others who were responsible for the maintenance of Delta levees at the time of the earthquakes. Summaries of these investigations are given in subsequent sections.

5.82 The August 6, 1979 Coyote Lake Earthquake

A moderate earthquake (M = 5.9) occurred at 10:05 PDT on August 6, 1979, in the Central California coast region. This event was located on the Calaveras Fault and induced Modified Mercalli Intensities of about II in the Delta Region. This intensity corresponds to a peak ground acceleration of less than 0.01g.

Table 5-2: Ground Damage in Delta
Reported by Finch (1985)

Map No.	Earthquake	Delta Island or Tract	Distance to Epicenter (mi)	Damage
1	Coyote Lake (9/6/79) M = 5.9	Mandeville	65	A 500-foot section of the west levee moved landward several feet. It was noticed independently by two people, and first seen minutes after the earthquake.
2	Livermore (1/24/80) M = 5.9	Bacon	15	A 250-foot landside rotational slip-out dropped several feet. This damage was cited by the 1980 DWR Delta seismicity report as possible earthquake-related damage.
3	Livermore	Empire	20	A 200-foot landside rotational slip-out dropped 6 inches. It was reported by a local resident and a DWR employee.
4	Coalinga (5/2/83) M = 6.7	Webb	150	A 500-foot crack opened along the levee crown up to 5 feet wide. Four or five landside rotational slip-outs caused a bulldozer to fall off the levee. Several eyewitnesses were present.
5	Coalinga	Webb	150	The "Garrett Well," an abandoned artesian well, and the site of seepage for many years, stopped following. The claim is supported by DWR photographs both taken before and after the earthquake.
6	Coalinga	Venice	150	A 500-foot crack opened on the landside toe of the levee and dropped from several inches to over 2 feet. The damage was noticed minutes after the earthquake.
7	Coalinga	Venice	150	An area of persistent seepage into a drainage ditch for many years. The seepage stopped after the earthquake.
8	Coalinga	Venice	150	Several cracks opened at the site of the 1982 levee break. One crack was 400 feet long and 10 to 20 feet deep. Another crack had water pouring out of it.
9	Coalinga	Venice	150	A 1000-foot crack ran along the levee toe. It was up to 3 feet wide and 10 to 15 feet deep.
10	Coalinga	Venice	150	At this site 14 wooden pilings popped up in a field that had been mowed the day before. The tops of pilings were evenly 9 feet above the ground surface. The pilings were the foundation of an abandoned horse barn.

Table 5-2: Ground Damage in Delta
Reported by Finch (1985)
(continued)

Map No.	Earthquake	Delta Island or Tract	Distance to Epicenter (mi)	Damage
11	Coalinga	King	160	The concrete floor of a shed cracked for a length of 25 feet and settle about 8 inches.
12	Pittsburg (6/5/83) M = 3.6	Webb	15	Several minor cracks were noticed at the Coalinga damage area. These cracks were at right angles to those produced by the Coalinga earthquake.
13	Morgan Hill (4/24/84) M = 6.2	Webb	60	Six parallel cracks one-inch wide and 75 feet long were noticed minutes after the earthquake. They were not present the day before the earthquake.
14	Morgan Hill	Webb	60	A 25-foot-long crack one-inch wide was noticed the same time as Site No. 13.
15	Morgan Hill	Venice	60	A pre-existing 25-foot-long crack lengthened 75 feet and the landside of the levee dropped 2 inches. This site was inspected by the island caretaker and DWR employees before and after the earthquake.

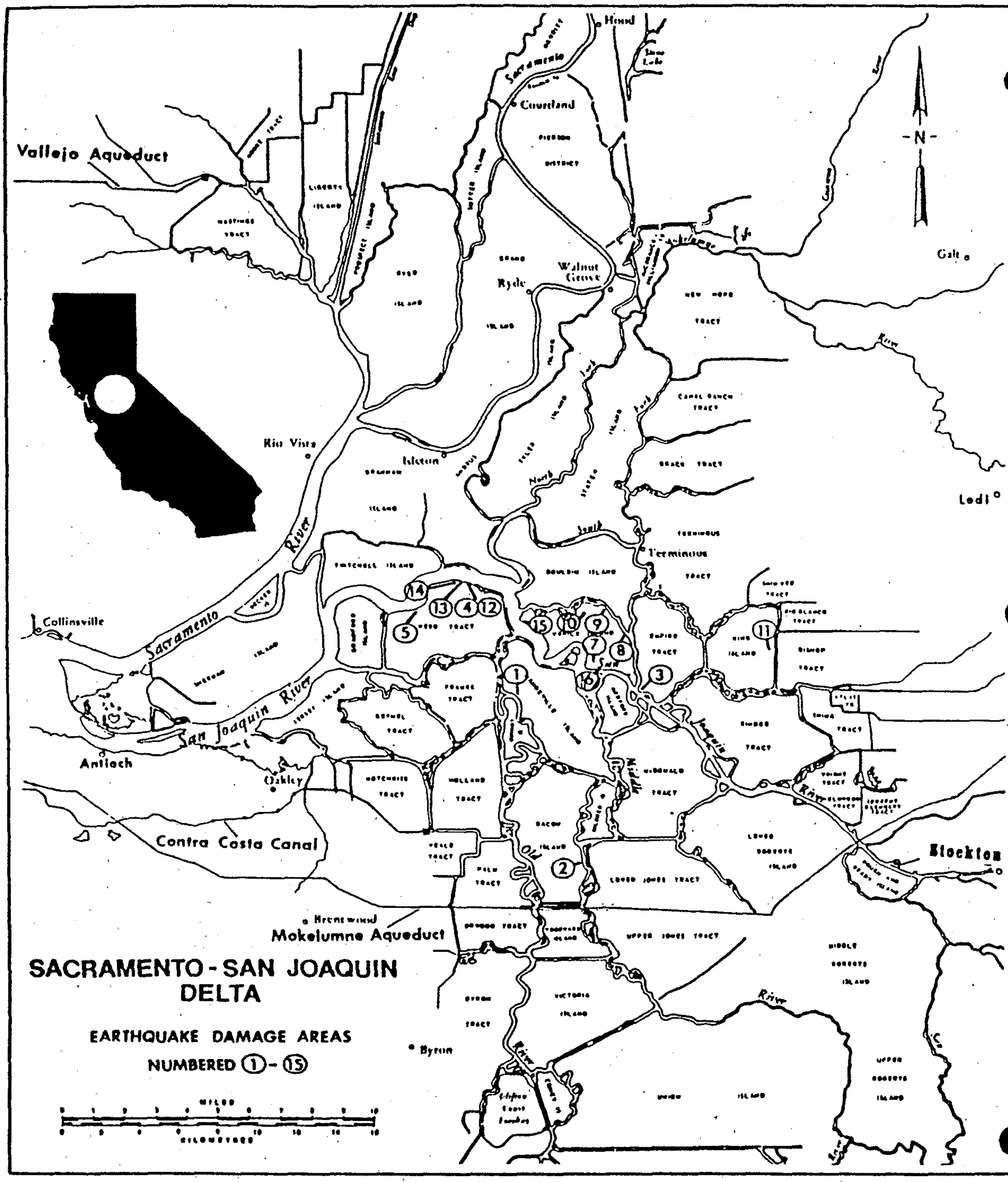


Figure 5-6: Locations of Earthquake-Induced Levee Damage Reported Between 1979 and 1984 (From Finch, 1985)

5.821 Mandeville Island

The only known report of levee damage in the Delta following the 1979 Coyote Lake Earthquake occurred at Mandeville Island, located approximately 65 miles away. Finch (1985) reported that a 500-foot section of the west levee move landward several feet. It was further reported that this movement was first observed only a few minutes after the earthquake and was noticed independently by two individuals. No confirmation of this event could be found.

5.83 The January 24 and 26, 1980 Livermore Earthquakes

The January 24 and 26, 1980 earthquakes, occurred in the east central coast ranges near Livermore. The January 24 event was a Magnitude 5.5 (UCB Seismographic Station) and originated 16 kilometers north of Livermore. On January 26, a Magnitude 5.8 event occurred approximately 6 kilometers north-northeast of Livermore. Both events were attributed to the Greenville Fault and induced Modified Mercalli Intensities of approximately IV-VII in and around the general Delta region. This would correspond to peak ground accelerations of approximately between 0.01g and 0.12g, and may represent one of the highest earthquake shaking induced in the Delta in historical times. During the January 24 event, the seismograph at the Department's Delta Pumping Plant, founded on rock, recorded a peak horizontal acceleration of 0.14g. This plant is located southwest of Clifton Court Forebay near the southwest edge of the Delta region.

The River News-Herald and Isleton Journal of January 30, 1980 reported no damage in the Delta area from the earthquakes. However, Finch (1985) reported that damage had occurred on Bacon Island and Empire Tract following the events.

5.831 Bacon Island

Finch (1985) reported that the east levee on Bacon Island experienced a 250-foot landside rotational slip-out which dropped several feet following the Livermore earthquakes. The site was approximately 20 miles away from the earthquake epicenters. The levee damage was also reported in a 1980 DWR Delta Seismicity Hazards Report as possible earthquake-related damage and provided the following quote from then Director Ron Robie reporting on January 1980 flood conditions:

> "The January 23, 1980 5.5 magnitude earthquake shook the already battered Delta levees. This created large waves, and threatened to destroy hard-won progress made during a week-long battle. A series of aftershocks followed adding to anxieties, but visual inspections did not show any significant earthquake-related damage. A 250-foot slip was detected on Bacon island midway on the east levee, but it has not been determined if the crack had been caused or only aggravated by the disturbance. Department engineers feared that the earthquake might be the last straw that would lead to structural damage to the substandard levee foundations and to extensive levee failure."

As the quote indicates, the Delta was experiencing flood conditions and high water at the time of the 1980 earthquakes. During periods of high water, it is not at all unusual for Delta levees to experience landside slumping. Consequently, the 250-foot slump cannot be definitively attributed to the earthquake sequence.

In 1991, interviews were made with Mr. Kaysor Shimasaki, Bacon Island Superintendent. Mr. Shimasaki was the original source for the report of the 250-foot slip, but he stated in 1991 that he was uncertain whether the damage was pre-existing or earthquake-related.

5.832 Empire Tract

Finch (1985) reported that the west levee on Empire Tract experienced a 200-foot landside rotational slip-out which dropped several feet following the Livermore earthquakes. This site was also approximately 20 miles away from the earthquake epicenters and is located near the edge of a levee test section researched by the Department in 1961-62. According to the 1987 USACE Delta levee liquefaction study, this site has a levee fill of silty sand and silty peat overlying about 15 to 20 feet of soft peat. Below the soft peat is a layer of silty sand approximately 10 to 14 feet thick. Although no SPT data are available, penetration test data are available for one inch drive samplers. This information led the USACE (1987) to conclude that some of the sand layers would liquefy for a peak ground acceleration of 0.05g.

It should be noted that, as for the slip at Bacon Island, the reported slip at Empire Tract occurred during a month of heavy rainfall and high water. Furthermore, six islands or tracts (Dead Horse, Holland, Lower Jones, Prospect, Upper Jones, and Webb) were inundated during the 1980 flood season. Webb Tract flooded on January 18, 1980, six days before the January 24 earthquake. Consequently, the slips on both Bacon and Empire Tracts may be more related to flood stages in the San Joaquin River than to the 1980 Livermore Earthquakes.

5.84 The May 2, 1983 Coalinga Earthquake

The 1983 Coalinga Earthquake occurred at 4:43 PDT on May 2, 1983, near the town of Coalinga in Central California. The main shock was assigned a Richter Magnitude of 6.7 by the University of California Berkeley Seismographic Station. According to Stover (1983), this event subjected areas in and around the Delta to Modified Mercalli Intensities between II and IV. This corresponds to peak ground accelerations of about 0.02g. Finch (1985) reported eight case histories of levee damage for Webb, Venice, and King Island tracts following this earthquake. No other report of levee damage in the Delta could be found for this earthquake.

5.841 Webb Tract

Webb Tract, located approximately 150 miles away from the epicenter of the 1983 Coalinga Earthquake, is reported to have experienced two incidents of levee damage due to the Coalinga Earthquake. According to Finch (1985):

> "A 500-foot long crack opened along the levee crown up to 5 feet wide. Four of five landside rotational slips-outs caused a bulldozer to fall off the levee."

Recent discussions (1991) with one of the original eyewitness, Mr. Larry Reedy, indicate that in actuality the bulldozer "just nosed over a few inches into a crack. However, since it was muddy it was necessary to pull it out of this position." At the time of the earthquake, a crew with a suction dredge was reinforcing this already unstable reach of levee by dredging material from the waterside and placing it on the landside slope. Since the placement of fill on soft levees and their foundations generally cause at least limited slumping and/or cracking, it would appear that there would be incipient cracks from the fill placement alone, and that the earthquake shaking simply triggered the cracking and slumping to surface.

This levee site on Webb Tract had been a site of previous instability. According to the Reclamation District Engineer, Mr. Ken Kjeldsen, about 1000 tons of fill material was placed near this location in 1981 to fill what he referred to as a "dinosaur pit." This feature is described by Mr. Ken Kjeldsen as a linear feature parallel to the levee toe, about 40 to 50 feet wide and 600 feet long and at least 35 feet deep. The soil in the "pit" is described to have a pea soup consistency. The district engineer said he was unaware of any earthquake-induced ground cracking due to the Coalinga Earthquake.

The other incident of reported damage relates to a seep on the north side of Webb Tract and known locally as Garrett's Well (named after a prior owner of the tract). This site had continuous seepage problems and unsuccessful attempts were made in the past to stop the flow. Finch (1985) reports that the flow stopped following the Coalinga Earthquake. Messrs. Ken Kjeldsen and Larry Ready indicate that this was true and that it was most probably due to the earthquake. However, this "damage" seems to be better characterized as an improvement.

5.842 Venice Tract

Venice Tract, also located approximately 150 miles away from the epicenter of the 1983 Coalinga Earthquake, is reported to have experienced five incidents of ground damage due to this event. According to Finch (1985):

a. A 500-foot crack opened on the landside toe of the levee and dropped from several inches to over 2 feet.

b. An area of persistent seepage into a drainage ditch for many years stopped after the earthquake.

c. Several cracks opened at the site of the 1982 levee break. One crack was 400 feet long and 10 to 20 feet deep; another crack had water pouring out of it.

d. A 1000-foot crack ran along the levee toe. It was up to 3 feet wide and 10 feet deep.

e. Fourteen wooden piles popped up in a field that had been mowed the day before. The tops of the piles were evenly 9 feet above the ground surface. The piles were the foundation of an abandoned horse barn.

The island caretaker, Mr. Roberto Ponce, originally reported these incidents and reiterated them during a 1991 interview. However, other pertinent facts about this case seem to indicate that perhaps not all of the damage was earthquake-related. Venice Island flooded in the winter of 1982 and remained flooded at the time of the Coalinga Earthquake. The removal of the water was completed in the summer of 1983. According to the Reclamation District Engineer, Mr. Ken Kjeldsen, there were many unstable levee reaches at that time and erosion of the interior slopes of the levees was severe. Along some levee reaches the levee crown had been completely eroded. The District Engineer said he was not aware of any damage that was definitively induced by the earthquake. And, as for the "Garrett Well" on Webb Tract, one of the incidents listed above refers to a cessation of seepage. This should be considered an improvement rather than "damage."

On July 9 and September 10, 1991, Department staff inspected the locations of levee damage reported above. On July 9, one levee section had been recently repaired about a week earlier to remediate levee cracking and slumping. However, between this visit and the second visit in September, the levee section had slumped again and required an additional repair consisting of adding additional fill on the levee crest together with a buttress fill on the landside toe. During the September 10, 1991 inspection, other levee reaches were under repair, including some of the sections where cracking had been reportedly caused by the Coalinga Earthquake.

5.843 King Island

Finch (1985) reports that the concrete floor of a shed cracked for a length of 25 feet and settled about 8 inches. Recent discussions (1991) with Mr. Edward Marchetti, owner of the shed, revealed that the crack had been there since before the earthquake, although it might have become slightly wider after the earthquake.

5.85 June 6, 1983 Pittsburg Earthquake

Finch (1985) reported that a Magnitude 3.6 earthquake occurred near Pittsburg on June 6, 1983 and caused cracking at Webb Tract approximately 15 miles away. This is a relatively small magnitude and is generally not considered capable of producing significant damage. No other accounts of damage could be found for this event.

5.851 Webb Tract

The cracks reported on Webb Tract were at the same unstable levee section where cracking and slumping was reported following the 1983 Coalinga Earthquake. These cracks were reported as minor cracks occurring at right angles to the previous cracks reported to have been attributed by the Coalinga event. According to Mr. Ken Kjeldsen of Kjeldsen and Sinnock, cracks of this size and configuration are common on Delta levees and should be expected to be found at marginally stable levee sections regardless of earthquake shaking.

5.86 The April 24, 1984 Morgan Hill Earthquake

On April 24, 1984, a moderate earthquake occurred on the Calaveras Fault to the east of San Jose. The earthquake was felt throughout central California and had a Richter Magnitude of 6.2. The epicenter of the main shock was located 5 kilometers west-southwest of Mount Hamilton and about 65 kilometers northwest of the junction of the Calaveras and San Andreas Faults (Bakun, et al., 1984). This resulted in Modified Mercalli Intensities between II and IV within and around the Delta region, corresponding to peak ground accelerations of approximately 0.02g.

As for other earthquakes between 1979 and 1984, the only report of significant earthquake-induced damage in the Delta comes from Finch (1985). This study states that three incidents of levee damage resulted from this earthquake on Webb Tract and Venice Island.

5.861 Webb Tract

Finch (1985) reported that six parallel cracks one inch in width and 75 feet long were observed along the north levee of Webb Tract. The report stated that they were noticed minutes after the earthquake and were not present the day before the earthquake. Another one inch wide crack with a length of 25 feet was also noted at another site along the north levee at the same time. Due to the marginal nature of the reported damage, these two locations were not investigated for the current evaluation.

5.862 Venice Island

Finch (1985) reported that a pre-existing 25-foot-long crack lengthened to 75 feet and the landside of levee dropped 2 inches. It was also reported that this site was inspected by the island caretaker and DWR employees before and after the earthquake. These persons, Messrs. Jeff Northrup and Roberto Ponce were interviewed in 1991 and confirmed this account for this investigation.

5.9 1989 LOMA PRIETA EARTHQUAKE

The Loma Prieta Earthquake of October 17, 1989, occurred at 5:04 p.m. local time, when a segment of the San Andreas Fault northeast of Santa Cruz, California, ruptured over a length of approximately 28 miles (Seed, et al., 1990). The epicenter was located approximately 10 miles northeast of Santa Cruz and approximately 50 miles from Clifton Court Forebay in the south Delta. The Richter Magnitude (M_L) was determined to be 7.0 by the University of California Berkeley Seismographic Station and was calculated to have an average surface wave magnitude (M_S) of 7.1 by the U. S. Geological Survey.

Figure 5-7 presents an isoseismal map for the 1989 Loma Prieta Earthquake. Within and around the Delta region, the Modified Mercalli Intensity was generally about V, corresponding to peak ground accelerations between 0.02g and 0.05g. At Clifton Court Forebay, a seismograph located on mineral soils downstream from the dam recorded a peak ground acceleration of 0.08g. At the Department's Delta Pumping Plant, founded on rock and located about two miles southwest of Clifton Court, seismographs recorded peak ground accelerations of about 0.06g.

There were a few reports of damage in the Delta following the Loma Prieta Earthquake. However, only one report of ground damage was found and this was at McDonald Island. There were other reports of building and/or structural damage on Brannan-Andrus Island, Lower Jones Tract, and in the Suisun Marsh

5.901 McDonald Island

The ground damage which occurred at McDonald Island was reported by a team of inspectors from the Department's Division of Flood Management (Coe, 1989). The report described cracking at seven locations along the island levee and a "sinkhole" at one site. Most of the cracks were less than one inch in width and less than 70 feet long. One crack was eight inches wide, one foot deep and 20 feet long. The "sinkhole" was approximately 100 feet inboard of the landside levee toe. A vortex was also reported to have appeared in Middle River near the location of the levee cracks. Personal communication in 1991 with Jeff Northrup, a member of the reporting team, confirmed the report.

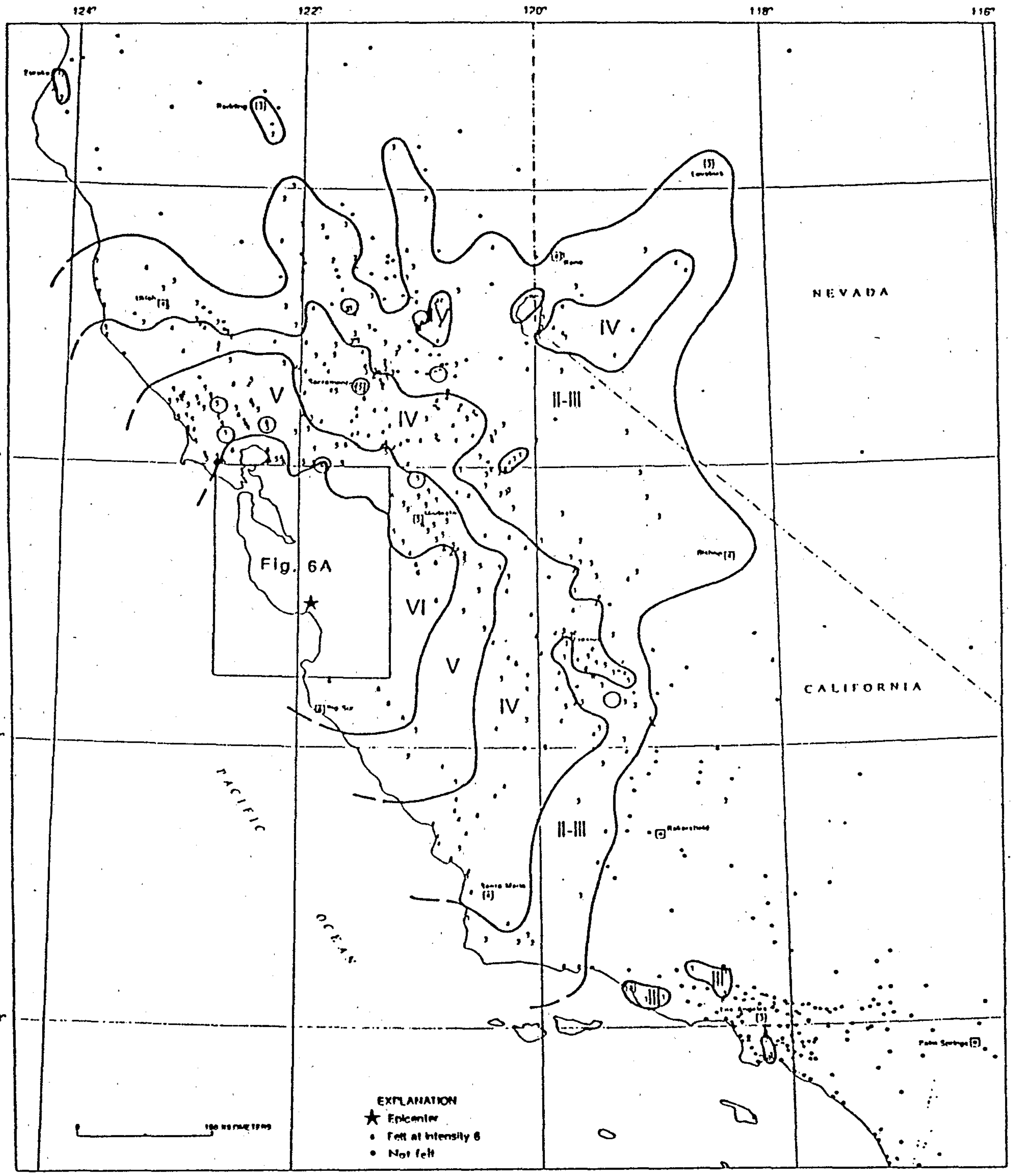


Figure 5-7: Isoseismal Map for the 1989 Loma Prieta Earthquake (From Stover, 1990)

The Pacific Gas and Electric Company also reported that a brace for a compressor was damaged on McDonald Island. (Gamble, 1991).

5.902 Brannan-Andrus Island

In addition to reports of minor damage to sewer pipes, it was reported that in the town of Isleton on Brannan-Andrus Island, the City Hall was damaged by the earthquake. The City Hall is a two-story structure with an unreinforced masonry first floor and a wood frame second floor, with a brick veneer exterior. The site is near, or perhaps on, the old alignment of the backfilled Jackson Slough.

Local officials reported that major structural damage had probably occurred following the earthquake. However, inspectors for the Federal Emergency Management Agency reported only minor damage to the exterior veneer on the second floor. Consequently, this prompted further inspections by the Sacramento County Department of Public Works, and a structural engineer hired by the town of Isleton. It was concluded that although the building did not meet earthquake standards prior to the earthquake, it was inconclusive whether the building was structurally weakened by the earthquake.

The level of shaking at Isleton from the Loma Prieta Earthquake is considered to have been relatively low and consistent with the damage intensities reported above. Discussions with Mr. Leonard Maxey, Superintendent of Public Works, City of Isleton, and Mr. Clyde Brandt, an Isleton building inspector, revealed that the building had pre-existing cracks and that it would be difficult to identify those which were caused by the earthquake. Mr. Maxey said that passing trucks probably caused as much shaking as the earthquake. They also indicated that the level of shaking they felt in 1989 was less than that felt during the 1980 Livermore earthquake for which there was no reported damage. A local antique store containing numerous fragile glassware and pottery on shelves is located near the City Hall. Ms. Ida Pucci, the store owner, reported essentially no damage (one small figurine fell off a shelf).

Despite the foregoing, the City Hall has been closed and the city is seeking public funds to repair the structure.

5.903 Lower Jones Tract

The Rindge Elementary School on Lower Jones Tract was reported to have been damaged by the Loma Prieta Earthquake.

Six to eight inches of settlement and cracking of the building was originally reported by the Lodi News Sentinel. However, later newspaper reports say that the settlement may have been pre-existing. The school was flooded later that winter and abandoned. It is now a farm labor camp.

5.904 Suisun Marsh

Within the Suisun Marsh at the Montezuma Slough Salinity Control Structure, it was reported that the Loma Prieta Earthquake caused the walkway leading from the levee to the Boat Lock Structure to be displaced by several inches. The boat lock operator, Mr. Bell, also reported that he was thrown to the ground by the earthquake and that his pick-up truck, which was parked on the east embankment, bounced southward about a foot. Although the control structures within the channel are founded on stiff sands and clays, the levee is a recent setback fill constructed on approximately 30 feet of soft peat.

There are inconsistencies with the above reports, however. This site is located approximately 75 miles from the epicenter of the Loma Prieta Earthquake and no other accounts of significant ground motions were found for this area. In addition, a 1991 discussion with the former Resident Engineer at the time of construction, Mr. Don Mitchell, revealed that the walkway had been built with about 4 inches of offset. Another boat lock operator, Mr. Ray Ballesteros (presently a DWR construction inspector) also said during a 1991 interview that he remembered seeing the offsets in the walkway before the earthquake.

5.10 DISCUSSION

The Sacramento-San Joaquin Delta has been shown in the previous sections to have not sustained even moderate earthquake shaking within the last century. The foregoing sections also show that there is no conclusive evidence that significant earthquake-induced levee damage has occurred within the Delta during this time period. The case histories reported by Finch (1985) of levees slumping several feet following the occurrence of relatively distant earthquakes are not supportable. These incidents generally occurred when other events were also occurring (e.g., high water from flood stages, island already eroded, or fill being added to marginal levee at the time of the earthquake). Therefore, such incidents cannot be attributed to earthquake shaking alone.

There is also the obvious question with regard to the levee slumps on Webb and Venice Islands, which were attributed to the 1983 Coalinga Earthquake 150 miles away: Why didn't these marginal levees also slump several feet during the 1989 Loma Prieta Earthquake, a larger and closer earthquake which produced stronger shaking in the general region?

There is some support that minor levee cracks, including some of the incidents reported by Finch (1985), may indeed have been induced and/or enlarged following an earthquake (e.g., levee cracks on McDonald Island following the 1989 Loma Prieta Earthquake). However, such events occur during high tides and other occasions without earthquakes.

6. GROUND RESPONSE ANALYSES

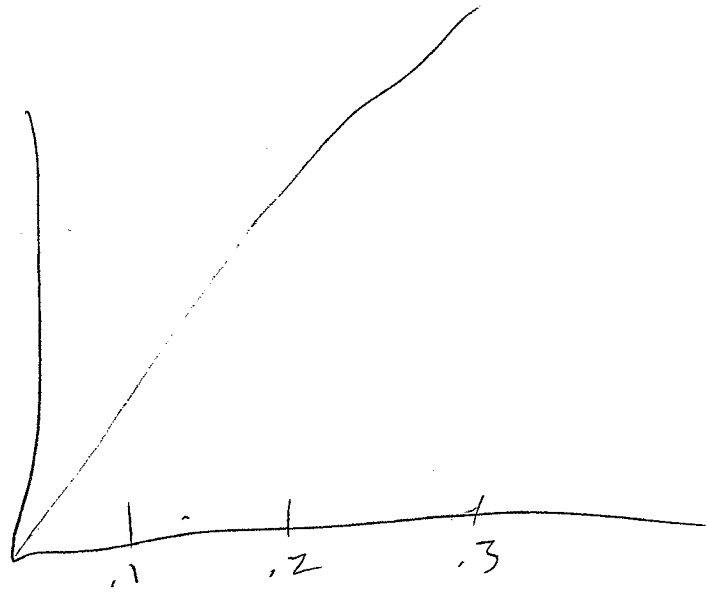
6.0 GENERAL

One of the most important lessons learned during the 1989 Loma Prieta Earthquake, was that soft soils may significantly amplify earthquake motions by factors as high as 3 to 5 times (see Figure 6-1). However, this may not necessarily be true if the soft soils in question are fibrous peats. Indeed, the only known earthquake records obtained from a recording site founded on peaty soils indicated severe attenuation or damping rather than amplification. These records (see Figure 6-2) were obtained from a Magnitude 4.5 earthquake located about 25 miles away (near Union Bay, Seattle). The motions were recorded through 58 feet of unconsolidated peat and were documented in a study by Seed and Idriss (1970). Because this effect may probably represent the largest unknown related to the assessment of the seismic stability of Delta levees, preliminary dynamic response analyses were performed. These response analyses assumed a range of properties for peat layers in order to determine the potential for such layers to either amplify or attenuate earthquake motions as they propagate to the levee surface.

The dynamic response analyses were performed using Program SHAKE90. The original version of this program, SHAKE, was developed by Schnabel, et al. (1972), and assumes that shear waves propagate vertically through horizontally-layered deposits. It is a commonly used one-dimensional response analysis and employs equivalent-linear properties to model the dynamic soil moduli and damping as a function of shear strain. The program uses the complex response method to solve the wave equation in the frequency domain.

6.1 RESPONSE MODELS FOR THREE TYPICAL DELTA SITES

In general, levees in the Delta consist predominantly of relatively loose, dredged sandy material which is often intermixed with other types of fill, including organic and inorganic silts and clays. Typical levee cross sections were previously shown in Figures 2-10 and 2-12 through 2-14. As shown in these figures, Delta levees are commonly founded on layers of soft organic soils. In the Sacramento-San Joaquin Delta, the thickness of these soft soils ranges between 0 and 60 feet, with thicknesses between 10 and 30 feet commonly found. The soft organic layer is often underlain by a medium dense to dense sand with varying amounts of clay.



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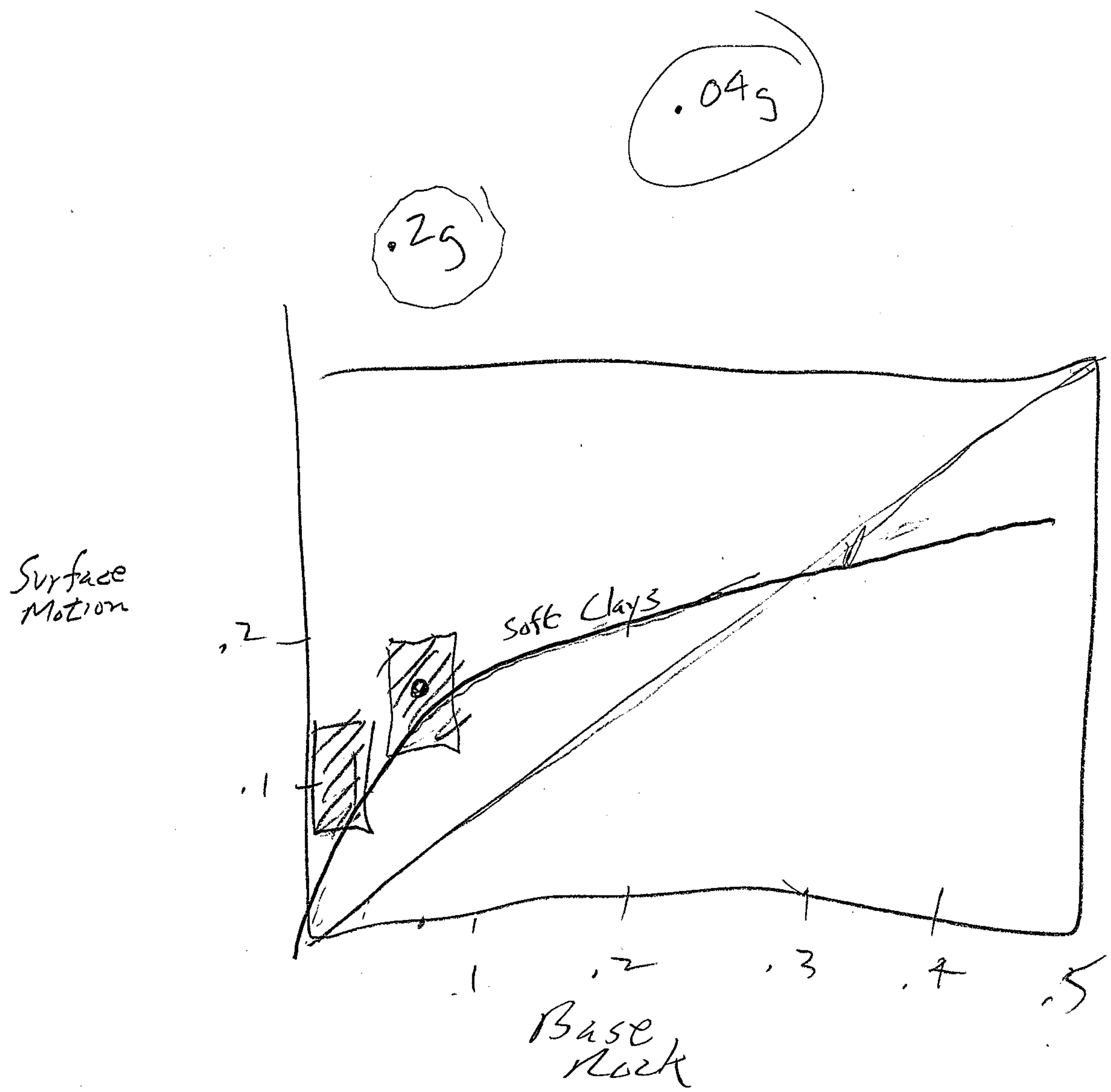

.04g
.2g
Surface Motion
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Soft Clays
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Base Rock

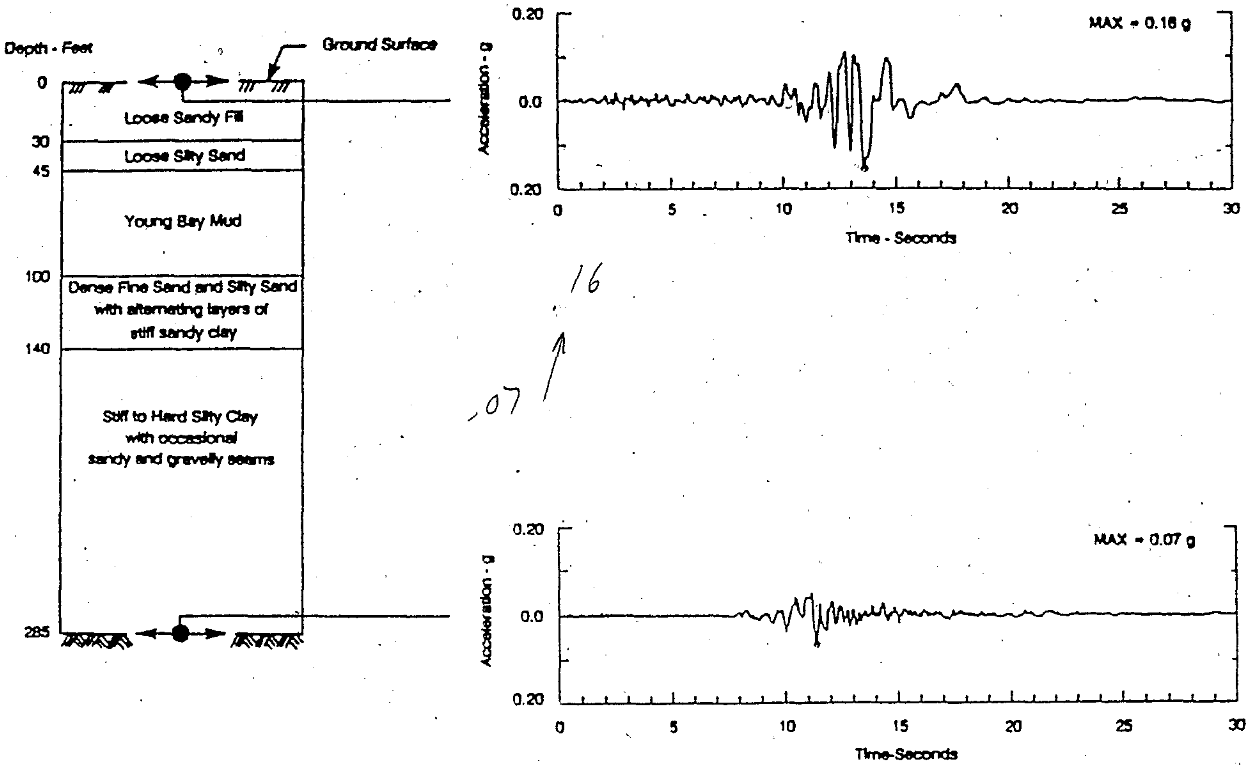
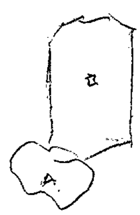


Figure 6-1: Schematic Soil Profile and Site Response at the Treasure Island Station (Seed et al., 1990)



San Fran

A.MP.
FACT ≈ 3

Union Bay

A.F. ≈ 0.1

A.F. ≈ 1 – 1.6

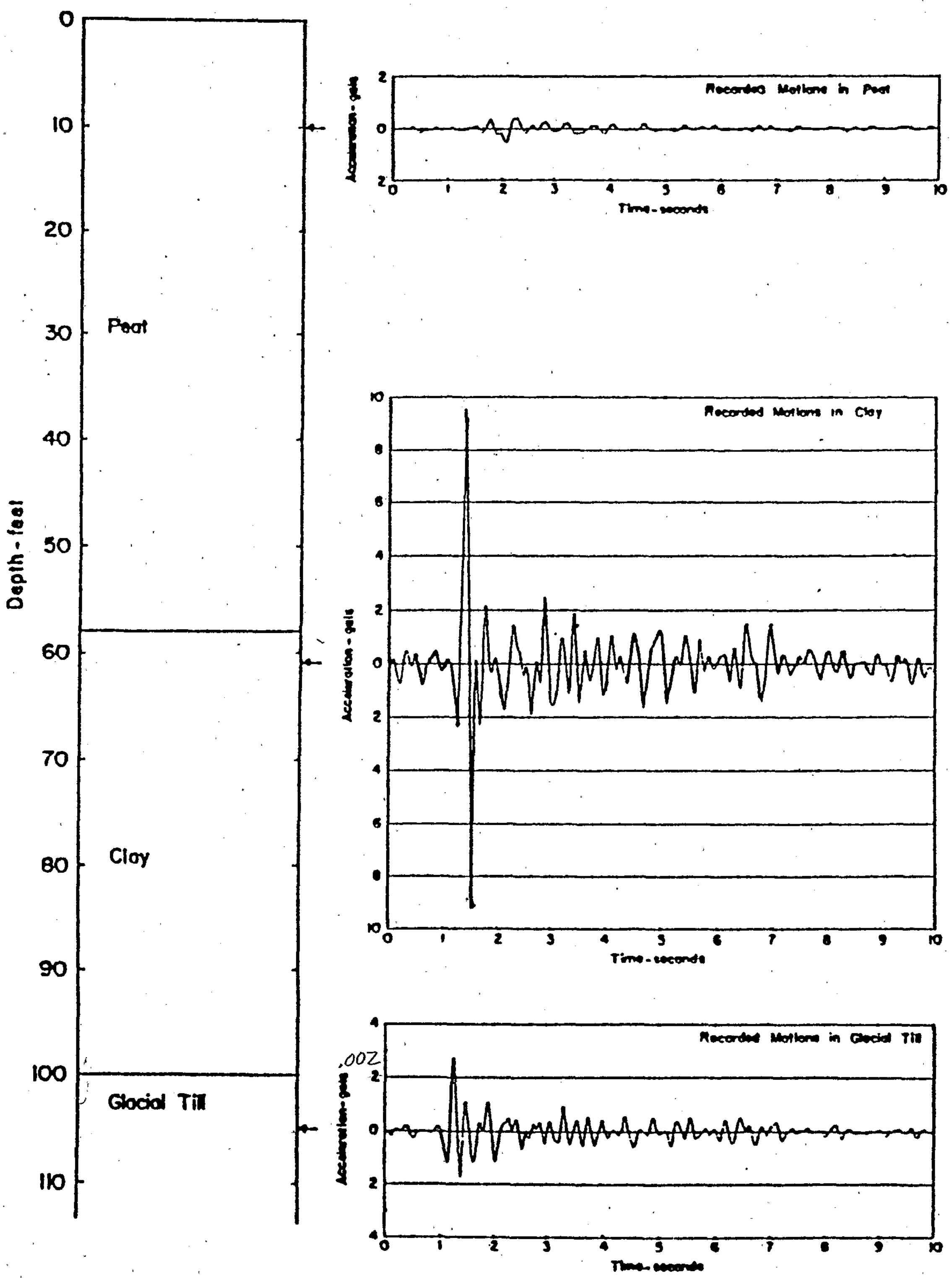


Figure 6-2: Recorded Motions from Union Bay Study (Seed and Idriss, 1970)

Three models were developed to perform dynamic response analyses. These models were developed from many geologic investigations to represent the general range in conditions found in the Delta. The models used in the SHAKE90 program are shown in Figure 6-3 and are intended to represent sections taken through the levee crown. Model A represents a thick sandy levee overlying foundation conditions which are commonly found in the western portion of the Delta where the organic soils are very thick. Models B and C represent smaller levees overlying foundation conditions which are commonly found in either the northern or southern portions of the Delta. The profiles for Models A and C are similar, in that both have predominantly sand underlying the soft organic layer. Model B is different, in that it has alternating layers of silts, clays and sands under the soft organic layer.

Information was developed for each of the models from available geologic investigations which were typically limited to the upper 100 feet or so. Levee and foundation organic soils were given layer thicknesses between four and eight feet in the model. Below the organic soils, layer thicknesses increased to a maximum thickness of 15 feet. The water table for each of the models was set at depths between 8 and 10 feet.

6.2 DYNAMIC SOIL PROPERTIES AT LOW STRAIN

Input for SHAKE90 requires assigning each layer a value of either maximum shear modulus (G_{max}) or maximum shear wave velocity (Vs_{max}). These values represent material properties at low strain (e.g., 10^{-4} percent). Values of G_{max} and Vs_{max} were selected for each of the different material types in the models as discussed below:

Sandy Levee

For the relatively loose sandy levee embankment in each model, maximum modulus values were calculated using a K_{2max} modulus factor of 40. This value of K_{2max} corresponds to a corrected Standard Penetration Test (SPT) blowcount of approximately 10, which appears to be typical for many Delta levees.

Foundation Peat

To represent the soft organic foundation materials, values of maximum shear wave velocity were estimated using published values for San Francisco Bay Mud (Seed, et al., 1990, and Dickenson and Seed, 1991), Peat (Harding Lawson Associates, 1990), and Delta silt (Harding Lawson Associates,

MODEL A

LAYER	THICKNESS (feet)	UNIT WEIGHT (pcf)	K2max	Gmax (ksf)	Vsmax (fps)	MATERIAL	MODULUS REDUCTION AND DAMPING CURVE	0 FT.
1	5	105	40	516	—	SAND		
2	5	105	40	893	—	SAND	M – SAND (CP<1.0 ksc) 1988	▽ 10 FT
3	5	115	40	1094	—	SAND		
4	5	115	40	1210	—	SAND	D – SAND 1970	
5	7	115	40	1336	—	SAND		27 FT.
6	5	80	—	—	375	PEAT	a) M – UNION BAY PEAT 1970 D – 2 X CLAY 1970	
7	5	80	—	—	375	PEAT		
8	5	80	—	—	400	PEAT	b) M – YOUNG BAY MUD 1988 D – CLAY 1970	
9	5	80	—	—	400	PEAT		
10	5	80	—	—	450	PEAT	c) M – CLAY (PI = 70) D – CLAY (PI = 70)	
11	8	80	—	—	450	PEAT		60 FT.
12	10	110	—	—	500	SILT/CLAY	M – YOUNG BAY MUD 1988	
13	10	110	—	—	550	SILT/CLAY	D – CLAY 1970	80 FT.
14	10	120	100	4883	—	SAND		
							M – SAND (CP=1–3 ksc) 1988	
15	15	120	100	5330	—	SAND		
							D – SAND 1970	
16	15	120	100	5820	—	SAND		120 FT.
17					2500			

MODEL B

LAYER	THICKNESS (feet)	UNIT WEIGHT (pcf)	K2max	Gmax (ksf)	Vsmax (fps)	MATERIAL	MODULUS REDUCTION AND DAMPING CURVE	0 FT.
1	4	105	40	461	—	SAND	M – SAND (CP<1.0 ksc) 1988	
2	4	105	40	799	—	SAND		▽ 8 FT.
3	4	115	40	979	—	SAND	D – SAND 1970	12 FT.
4	6	80	—	—	250	PEAT	a) M – UNION BAY PEAT 1970 D – 2 X CLAY 1970	
5	7	80	—	—	275	PEAT	b) M – YOUNG BAY MUD 1988 D – CLAY 1970	
6	7	80	—	—	300	PEAT	c) M – CLAY (PI = 70) D – CLAY (PI = 70)	32 FT.
7	6	120	—	—	700	CLAY	M – YOUNG BAY MUD 1988	
8	7	120	—	—	750	CLAY	D – CLAY 1970	
9	7	120	—	—	800	CLAY		52 FT.
10	6	125	75	3117	—	SAND	M – SAND (CP=1–3 ksc) 1988	
11	7	125	75	3341	—	SAND	D – SAND 1970	
12	7	125	75	3567	—	SAND		72 FT.
13	5	110	—	—	400	SILT/CLAY	M – YOUNG BAY MUD 1988	
14	5	110	—	—	400	SILT/CLAY	D – CLAY 1970	82 FT.
15	6	125	—	—	850	SILT	M – YOUNG BAY MUD 1988	
16	6	125	—	—	850	SILT	D – CLAY 1970	94 FT.
17	8	125	80	4621	—	SAND	M – SAND (CP=1–3 ksc) 1988	
18	9	125	80	4849	—	SAND	D – SAND 1970	
19	9	125	80	5079	—	SAND		120 FT.
20					2500			

MODEL C

LAYER	THICKNESS (feet)	UNIT WEIGHT (pcf)	K2max	Gmax (ksf)	Vsmax (fps)	MATERIAL	MODULUS REDUCTION AND DAMPING CURVE	0 FT.
1	4	105	40	461	—	SAND	M – SAND (CP<1.0 ksc) 1988	
2	4	105	40	799	—	SAND		▽ 8 FT.
3	5	115	40	992	—	SAND	D – SAND 1970	13 FT.
4	5	80	—	—	250	PEAT	a) M – UNION BAY PEAT 1970 D – 2 X CLAY 1970	
5	6	80	—	—	275	PEAT	b) M – YOUNG BAY MUD 1988 D – CLAY 1970	
6	6	80	—	—	300	PEAT	c) M – CLAY (PI = 70) D – CLAY (PI = 70)	30 FT.
7	5	115	70	2182	—	SAND		
8	5	115	70	2361	—	SAND	M – SAND (CP=1–3 ksc) 1988	
9	5	115	70	2528	—	SAND	D – SAND 1970	
10	5	115	70	2684	—	SAND		50 FT.
11	5	125	90	3660	—	SAND		
12	5	125	90	3873	—	SAND		
13	8	125	90	4134	—	SAND		
14	8	125	90	4433	—	SAND	M – SAND (CP=1–3 ksc) 1988	
15	8	125	90	4714	—	SAND	D – SAND 1970	
16	8	125	90	4979	—	SAND		
17	8	125	90	5231	—	SAND		
18	10	125	90	5500	—	SAND		
19	10	125	90	5785	—	SAND		120 FT.
20					2500			

Figure 6-3 Material Properties used in Program SHAKE Analysis for Models A, B, and C

1990). For Models B and C, whose levee thicknesses are 12 and 13 feet, respectively, the soft organic layer was assigned maximum shear wave velocities between 250 and 300 fps. For Model A, whose levee thickness is 27 feet, maximum shear wave velocities between 375 and 450 fps were selected.

Foundation Silt/Clay

Below the soft organic material, layers of silt, clay, or combinations of silt and clay were assigned Vs_{max} values based on published values for San Francisco Bay Mud (Seed, et al, 1990, and Dickenson and Seed, 1991). For these soils, Vs_{max} values between 400 and 850 fps were selected.

Foundation Sand

Sand layers at depth in the foundation were assigned G_{max} values based on typical SPT blowcount values and overburden effects. Typical SPT blowcount values for the foundation sands range between 25 and 60 blows per foot. Accordingly, K_{2max} modulus factors ranging between 70 and 100 were used to compute G_{max} values for these layers.

6.3 DYNAMIC STRAIN-DEPENDENT SOIL PROPERTIES

To model the non-linear dynamic soil moduli and damping characteristics (as a function of shear strain), different sets of modulus and damping curves were chosen to represent the different soil types for the three different models. These dynamic properties are fairly well established for some soil types, such as for sands, clays, and bay mud soils, but they are not well understood for some other soil types such as peat. Values of strain-dependent dynamic modulus and damping relationships were selected for each of the different material types in the models as discussed below:

Sandy Levee

The relatively loose sand levee embankment in each model was analyzed using the reduction of shear modulus (G/G_{max}) and damping values presented in Figures 6-4 and 6-5, respectively. The modulus reduction curve in Figure 6-4, represents sandy soils with effective confining pressures less than 1.0 tsf (Sun, et al., 1988). This curve was used along with the damping relationship for sands proposed by Seed, et al. (1970).

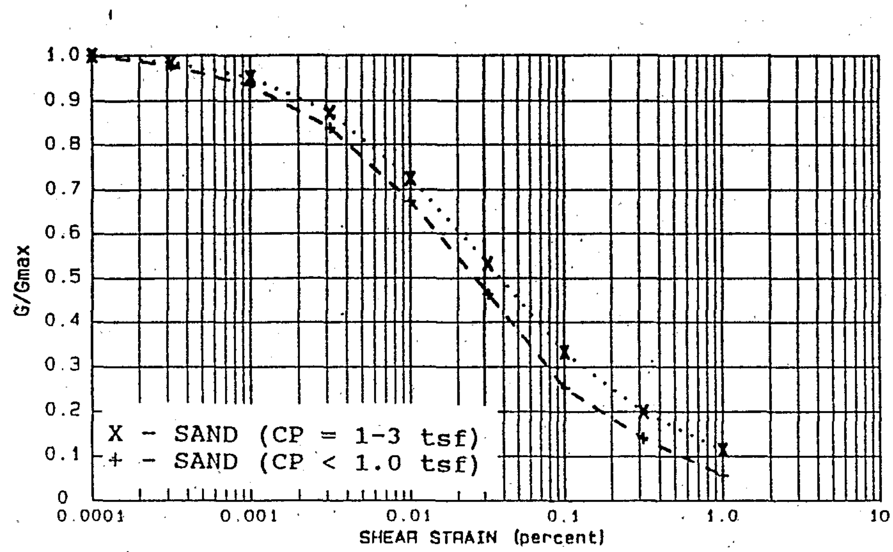


Figure 6-4 Normalized Modulus Reduction Curves for Sand (Sun et al., 1988)

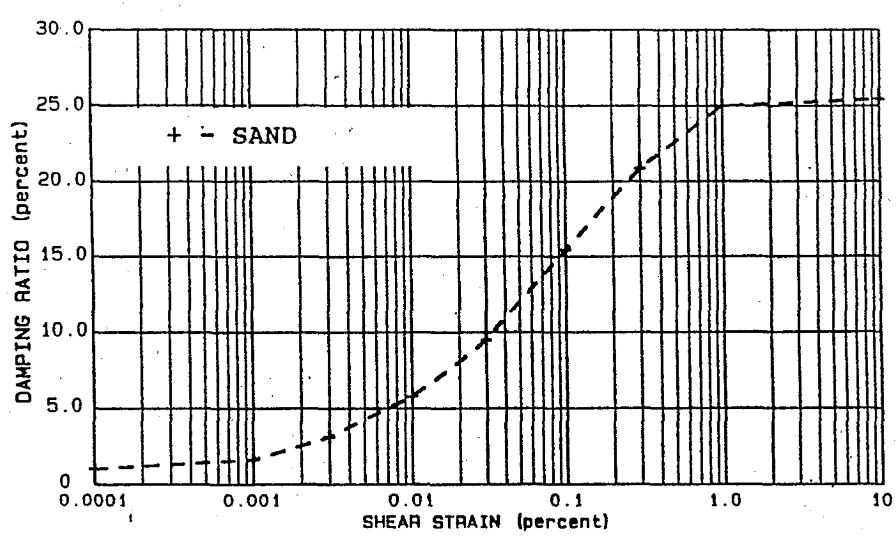


Figure 6-5: Damping Ratio Curve for Sand (Seed et al., 1970)

Foundation Peat

As previously discussed, there is a limited amount of data available concerning the modulus and damping characteristics for the soft organic "peat" soils in the Delta. The selection of these values, however, has the largest single effect on the results of the response analysis. Consequently, for this study, three different sets of material properties were chosen to represent the possible range in peat properties. The modulus reduction and damping curves chosen are shown in Figures 6-6 and 6-7.

Curves (a) were obtained from a study by Seed and Idriss (1970) for Union Bay Peat and were developed by back-calculating the response shown in Figure 6-2. In the current analysis, the damping values for Curve (a) were scaled down to 67 percent of their original values, as suggested by co-author Idriss, to more closely reflect actual conditions. Curves (a) will yield the lowest values of amplification in the response analysis.

Curves (b) in Figures 6-6 and 6-7 represent modulus and damping values proposed by Sun, et al. (1988), for young San Francisco Bay Mud. Use of these curves with SHAKE90 were shown to reasonably predict the amplification in ground motion observed at Treasure Island during the 1989 Loma Prieta Earthquake (see Figure 6-1).

Curves (c) in Figures 6-6 and 6-7 were obtained from Vucetic and Dobry (1990) to represent fat clay soils with a plasticity index of 70. This combination will yield the largest values of amplification in the response analysis. Also shown in Figures 6-6 and 6-7, are results of resonant column tests obtained from Harding Lawson Associates (1990) for Bouldin Island silt (Curves S) and peat (Curves P). The curves determined from field tests for silt appear to be in reasonably good agreement with curves for young San Francisco Bay Mud. The modulus curve determined from field tests on peat indicate values between the Union Bay peat and Bay Mud curves. Damping values from the field tests on peat are difficult to interpret, but lie within the Union Bay peat and fat clay curves (Curves a and c).

Information regarding the shear modulus and damping curves considered for peat is summarized in Table 6-1.

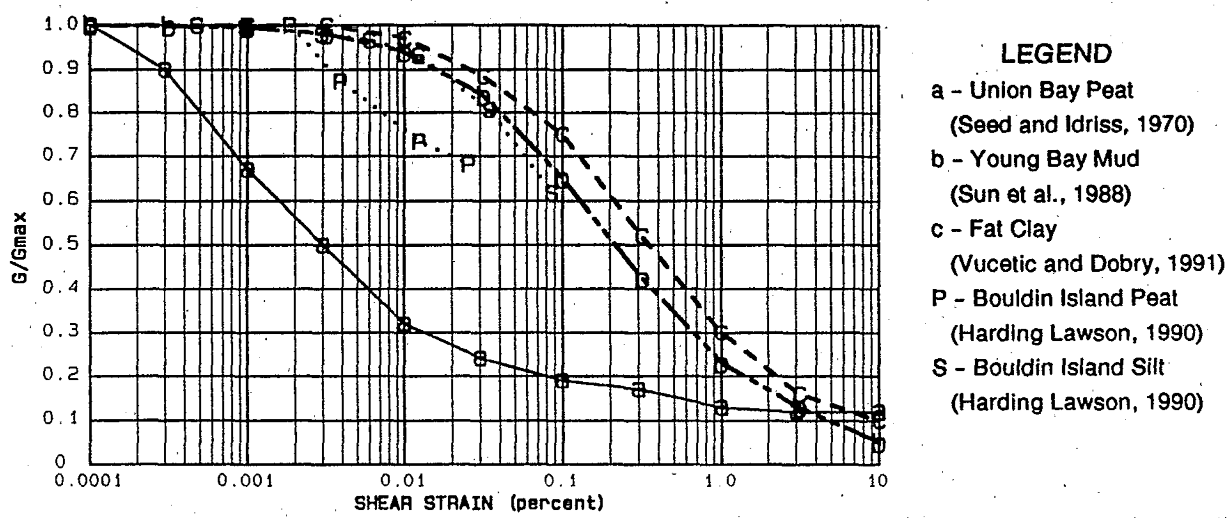


Figure 6-6: Normalized Modulus Reduction Curves for Peat

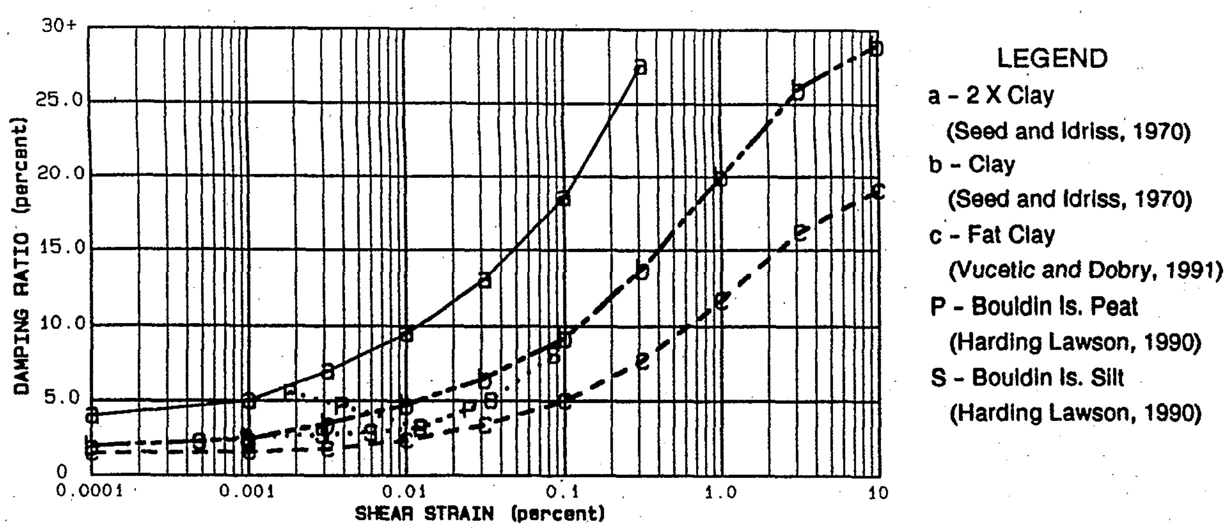


Figure 6-7: Damping Ratio Curves for Peat

Table 6-1: Shear Modulus and Damping Curves for Peat

CURVE	NORMALIZED MODULUS REDUCTION CURVE	SOURCE OF MODULUS CURVE	DAMPING CURVE	SOURCE OF DAMPING CURVE
a	UNION BAY PEAT	SEED AND IDRISS (1970)	2 X CLAY (SEE NOTE AT BOTTOM)	SEED AND IDRISS (1970)
b	YOUNG BAY MUD	SUN ET AL (1988)	CLAY	SEED AND IDRISS (1970)
c	FAT CLAY	VUCETIC AND DOBRY (1991)	FAT CLAY	VUCETIC AND DOBRY (1991)

NOTE: ORIGINALLY IN THE 1970 UNION BAY PEAT STUDY BY SEED AND IDRISS, A DAMPING CURVE EQUAL TO APPROXIMATELY THREE TIMES THE 1970 CLAY DAMPING WAS USED TO MODEL THE PEAT. FOR THE CURRENT STUDY, IT WAS RECOMMENDED BY DR. IDRISS THAT THE DAMPING CURVE SHOULD BE LOWERED AND A DAMPING CURVE EQUAL TO APPROXIMATELY TWO TIMES THE 1970 CLAY DAMPING WOULD BE MORE APPROPRIATE.

Foundation Silt/Clay

Below the soft organic material, layers of silt, clay, or combinations of silt and clay were modelled using the normalized modulus reduction curve for young Bay Mud developed by Sun, et al. (1988), and clay damping curve developed by Seed, et al. (1970). These curves are shown in Figures 6-8 and 6-9.

Foundation Sand

Sand layers in the foundation were modelled using the reduction of shear modulus (G/G_{max}) and damping values presented in Figures 6-4 and 6-5, respectively. The modulus reduction curve in Figure 6-4, representing sandy soils with confining pressures between 1 and 3 tsf proposed by Sun, et al. (1988), was used along with the damping relationship for sands proposed by Seed, et al. (1970).

6.4 EARTHQUAKE MOTIONS USED IN RESPONSE ANALYSES

Response analyses require time histories of earthquake base acceleration in order to load the soil column. However, there are a limited amount of recorded earthquake records available for the region surrounding the Sacramento-San Joaquin Delta. Consequently, earthquake records were selected from other locales and/or projects using the following criteria:

- Proximity to fault rupture.
- Earthquake magnitude.
- Foundation characteristics.

Three earthquake records were chosen for use in performing the response analyses to model earthquake magnitudes between 6.5 and 8.5. This range of earthquake magnitudes corresponds to major events that may occur in or near the Delta and possibly inflict damage. The characteristics of the three earthquake records are summarized in Table 6-2 and below:

Yerba Buena Island Record

The Yerba Buena Island record was obtained from the 1989 Loma Prieta Earthquake (M_s = 7.1). The larger E-W horizontal component at this recording site yielded a peak acceleration of 0.067g and was selected for use. The Yerba Buena motion was recorded on a rock outcrop very close to the Treasure

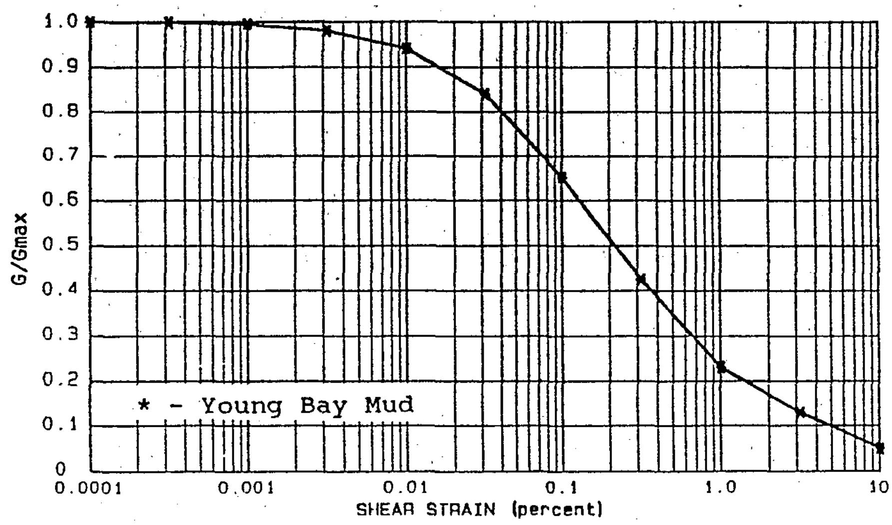


Figure 6-8: Normalized Modulus Reduction Curve for Clay (Sun et al., 1988)

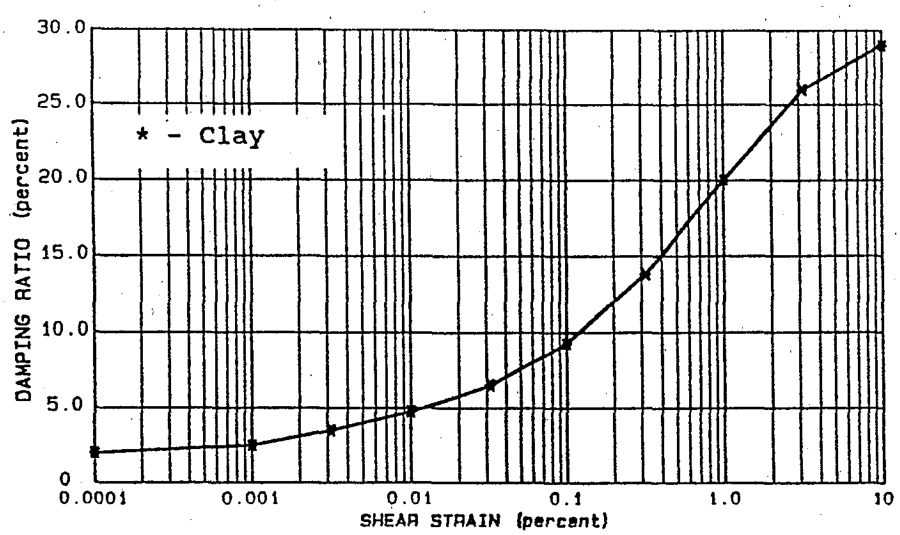


Figure 6-9: Damping Ratio Curve for Clay (Seed and Idriss, 1970)

Table 6-2: Accelerograms used in Dynamic Response Analysis

EARTHQUAKE	STATION LOCATION	ESTIMATED SOIL DEPTH (feet)	APPROXIMATE SOURCE DISTANCE (miles)	ORIGINAL PEAK ACCELERATION (g)	PREDOMINANT PERIOD (seconds)	PEAK ACCELERATION USED IN ANALYSIS (g)
IMPERIAL VALLEY 10/15/79 (M = 6.4)	MCCABE SCHOOL ARRAY 11	?	10	0.38	0.25	0.1, 0.15, 0.25
LOMA PRIETA 10/17/89 (M = 7.1)	YERBA BUENA ISLAND	ROCK	55	0.06	0.65	0.1, 0.15, 0.25
SEED-IDRISS SYNTHETIC EARTHQUAKE (M = 8.3)	----	ROCK	----	----	0.4	0.1, 0.15, 0.25

Island surface motion recorded on sand fill and Bay Mud. This is of particular interest because of the amplification of motion observed at Treasure Island (see Figure 6-1).

Figures 6-10 and 6-11 show the accelerogram and response spectrum (scaled to a peak 0.15g) for the Yerba Buena Island record. The predominant period is about 0.6 seconds. The dotted lines in Figure 6-11 represent mean and 84th percentile spectra computed by Seed, et al. (1974), for rock records.

Seed-Idriss Synthetic Earthquake Record

The Seed-Idriss Earthquake is a synthetic accelerogram intended to represent a rock motion produced during a Magnitude 8+ earthquake. This time history was developed because of the absence of any available records from such a large earthquake. Figures 6-12 and 6-13 present the accelerogram and response spectrum scaled to a peak 0.15g. The predominant period is about 0.4 second. The dotted lines in Figure 6-13 represent mean and 84th percentile spectra computed by Seed, et al. (1974), for rock records.

McCabe School Record

The McCabe School Array 11 motion was recorded on deep alluvial soils during the 1979 Imperial Valley Earthquake (M_L = 6.4). The peak acceleration of the S50W component was 0.38g. This motion was chosen because it was recorded on deep soils, not rock, and it may be more appropriate for application at a relatively shallow halfspace. Figures 6-14 and 6-15 present the accelerogram and response spectrum scaled to a peak 0.15g. The predominant period is about 0.25 second. The dotted lines in Figure 6-15 represent mean and 84th percentile spectra computed by Seed, et al. (1974), for stiff soil records.

Response analyses were performed using all three earthquake accelerograms scaled to have peak accelerations of 0.10g, 0.15g, and 0.25g at halfspace outcrops.

6.5 PARAMETRIC RESPONSE ANALYSES

In trying to overcome the unknowns regarding the strain-dependent properties of the peat, a range of properties was adopted. However, there remain additional unknowns regarding the properties of the soil column at depth. These unknowns include the types and properties of materials at depth, the depth to a rocklike halfspace, and the stiffness of the halfspace. In an attempt to

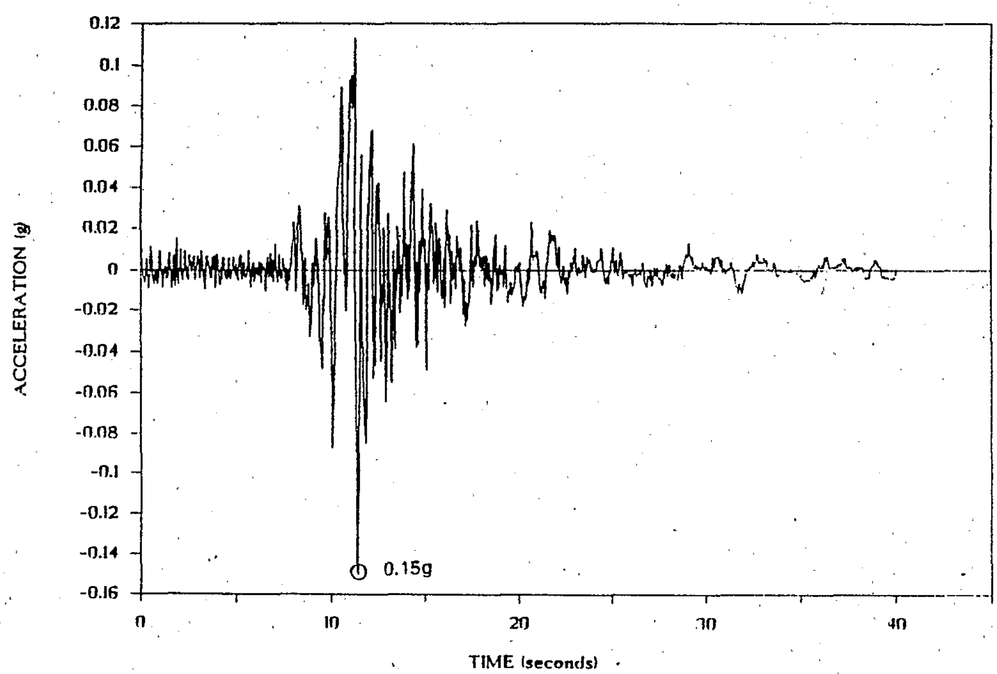


Figure 6-10: Yerba Buena Island Accelerogram

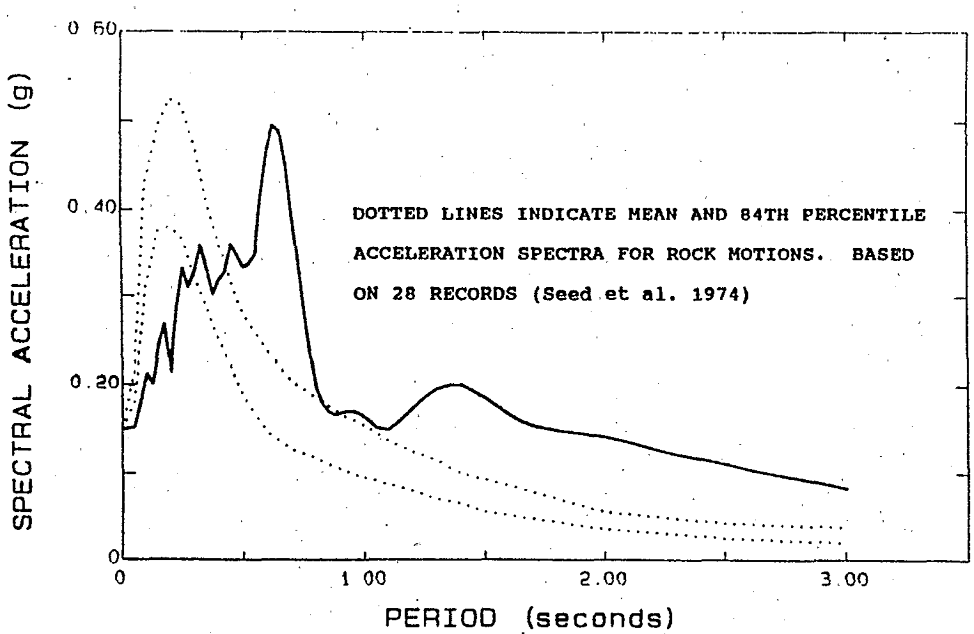


Figure 6-11: Yerba Buena Island Acceleration Response Spectrum

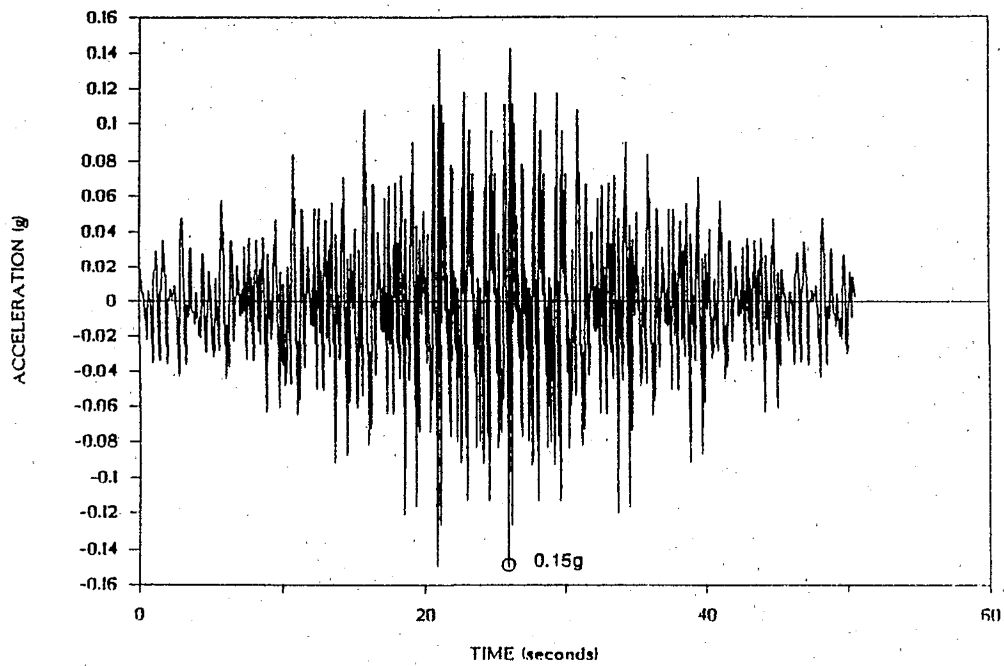


Figure 6-12: Seed-Idriss Accelerogram

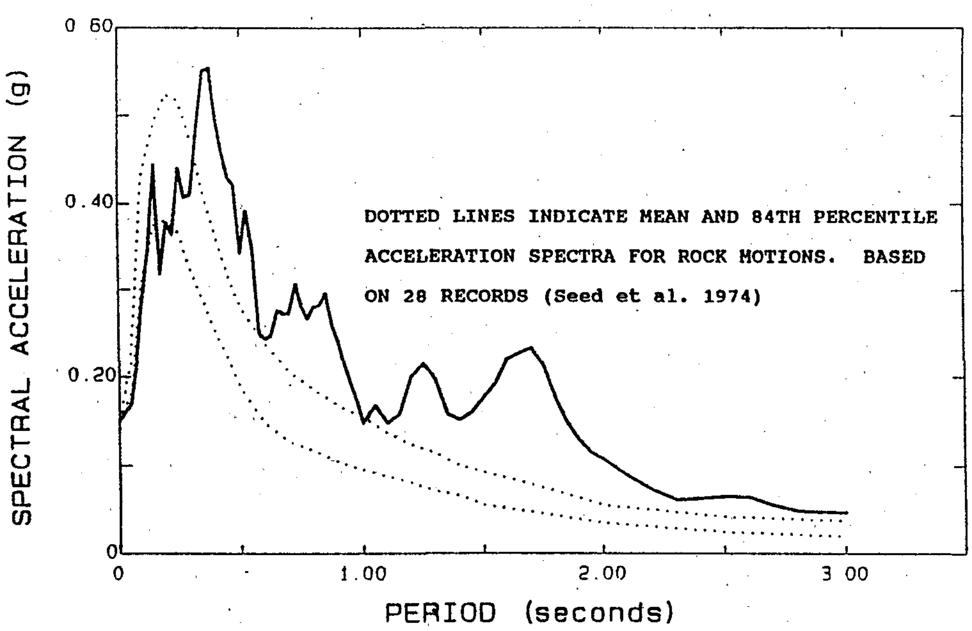


Figure 6-13: Seed-Idriss Acceleration Response Spectrum

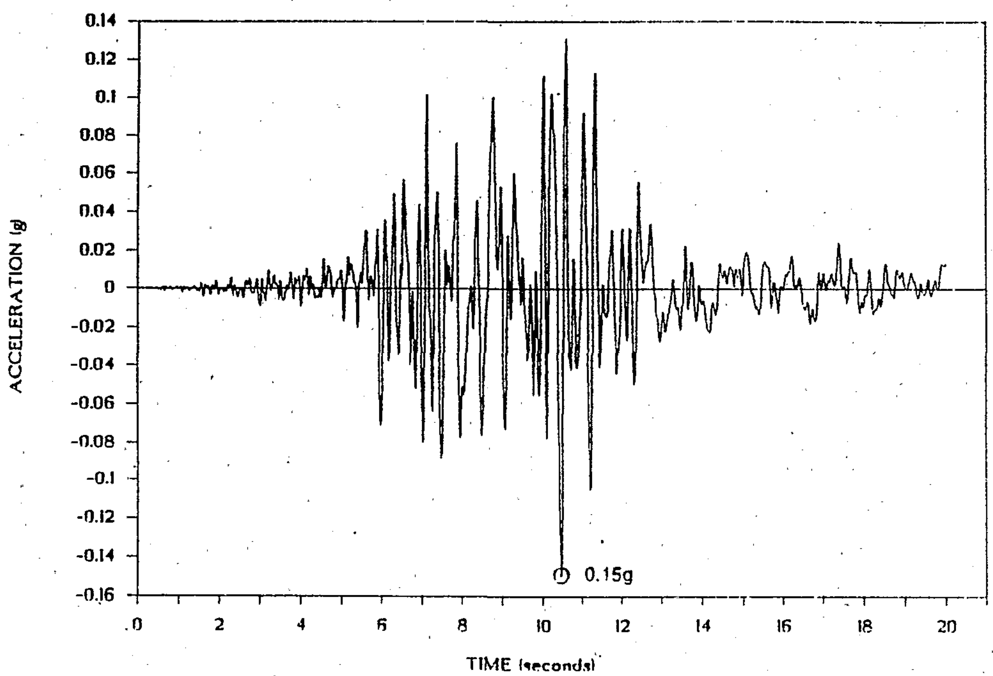


Figure 6-14: Mccabe School Accelerogram

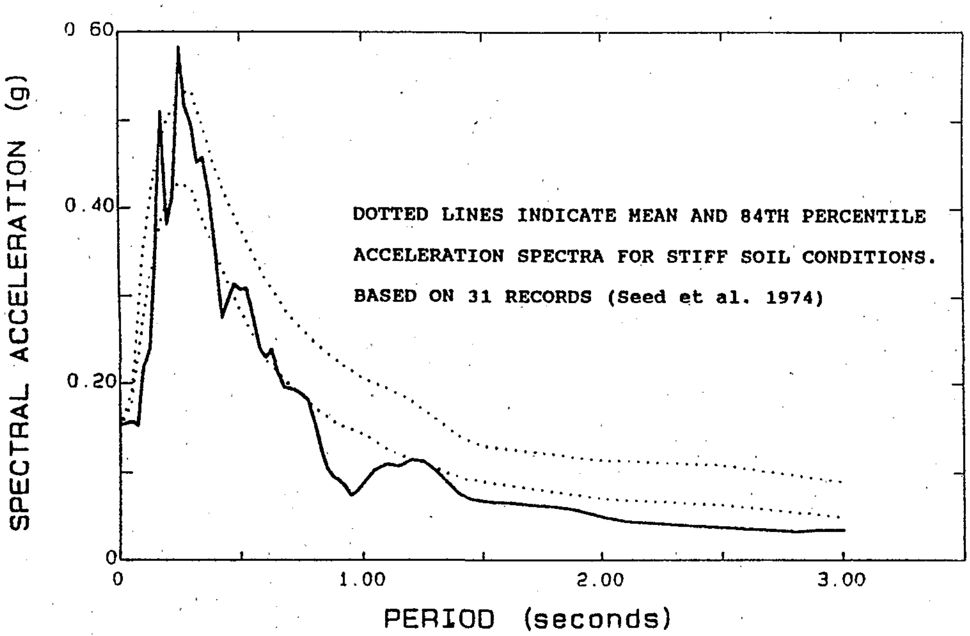


Figure 6-15: Mccabe School Acceleration Response Spectrum

resolve the possible effects these parameters might have, several parametric studies were performed to estimate the effect of varying some of the parameters. The parameters studied were:

- Height (or depth) of soil column. Soil columns were varied to have heights of 80, 120, 180, and 280 feet.

- The stiffness of the halfspace. The halfspace stiffness was varied to have Vs_{max} values of 1,500, 2,500, and 3,500 fps.

- Characteristics of deeper soil layers. A limited number of response analyses were performed using sand, clay, and alternating sand/clay layers to represent the properties of the soils below depths of 80 feet.

The parametric studies were all run using Model A assuming a modulus reduction curve for peat layers equal to that developed by Sun, et al. (1988), for Bay Mud (Curve b in Figure 6-6). The peat layers also employed the damping curve developed by Seed, et al. (1970), for clays (Curve b in Figure 6-7). The Yerba Buena Island accelerogram set to have a peak acceleration of 0.067g was used in all of the analyses. The results of the parametric studies are shown in Figures 6-16 through 6-22. These studies indicated the following:

- Figures 6-16 through 6-18 show the effect of soil column depth and the effect of the halfspace stiffness. These figures show that the computed surface peak ground accelerations increase as the height of the soil column increases. This is true for all three stiffnesses assumed for the halfspace, but becomes more pronounced as the halfspace stiffness increases. For example, the difference in computed surface accelerations for the four different column heights is only about 10 percent when the halfspace Vs_{max} equals 1,500 fps (see Figure 6-16). However, for halfspace Vs_{max} values of 2,500 and 3,500 fps, the computed differences in surface peak accelerations are about 35 and 45 percent, respectively (see Figures 6-17 and 6-18).

- Figures 6-19 through 6-21 present responses computed for a soil column height of 280 feet and a halfspace Vs_{max} of 2,500 fps. For these three different analyses, the soil below a depth of 80 feet was varied from sand to clay to intermixed sand and clay layers. However, as shown in Figure 6-22, the responses were essentially identical. This result suggests that the properties of the deeper soils were not very important to the overall response of

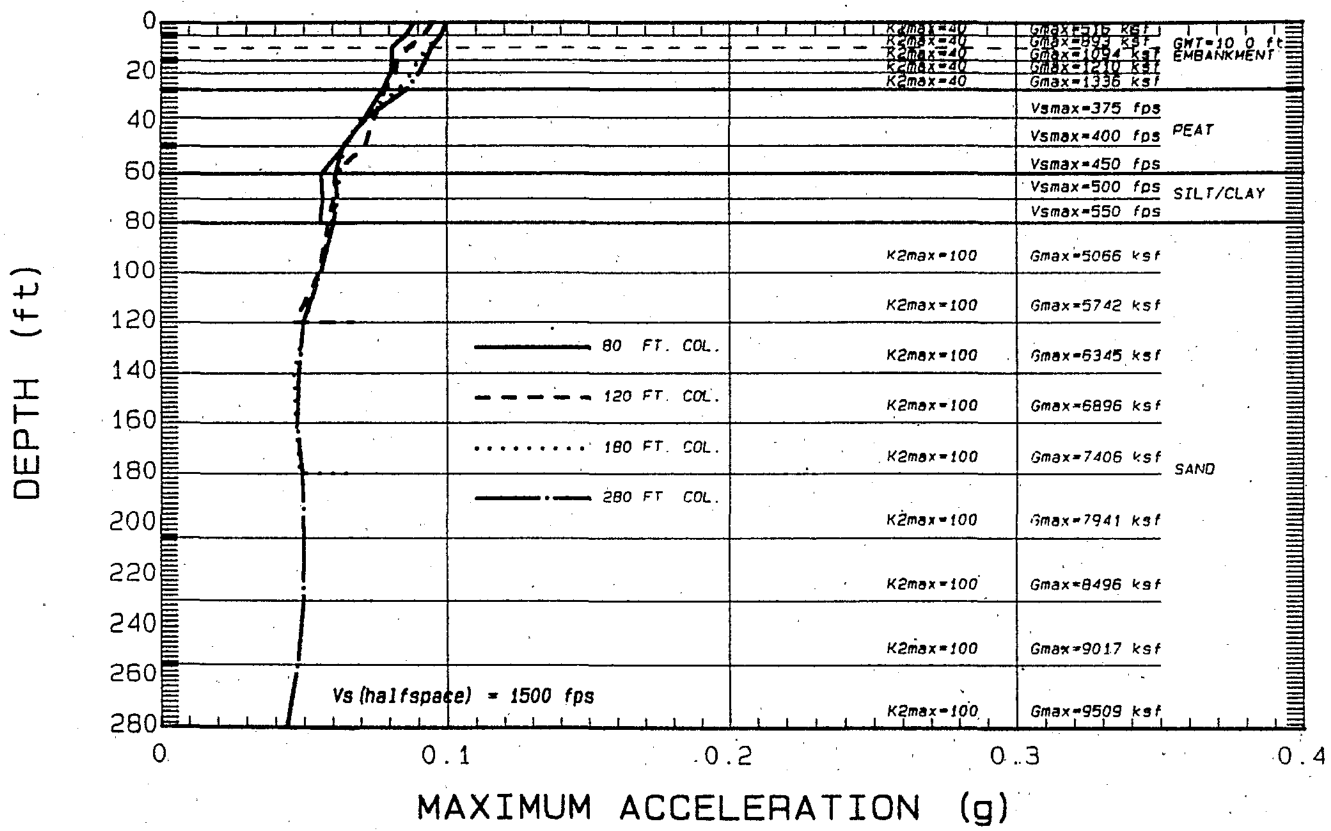


Figure 6-16: Model A Parametric Study - Effect of Varying Model Height. Vs_{max}(halfspace) = 1500 fps.

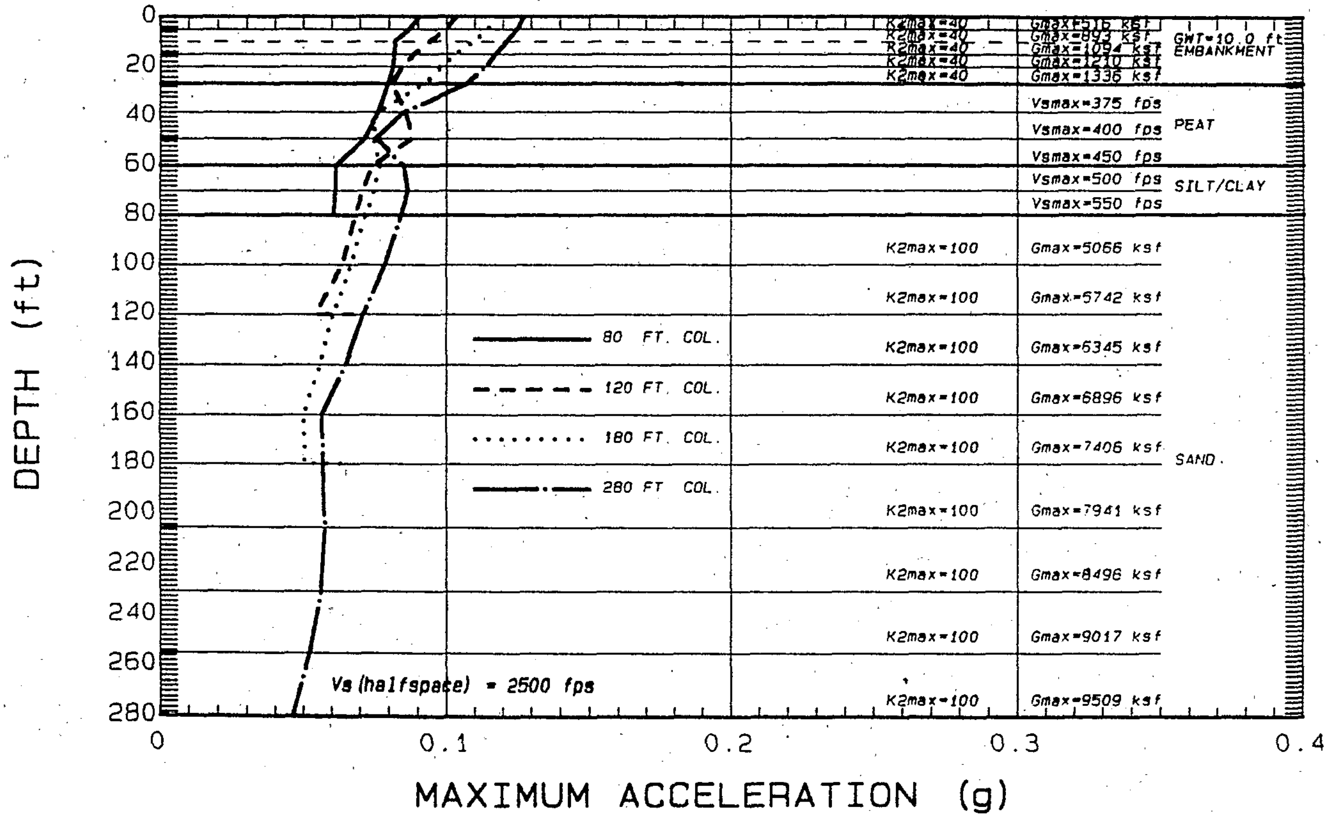


Figure 6-17: Model A Parametric Study - Effect of Varying Model Height. Vs_{max}(halfspace) = 2500 fps.

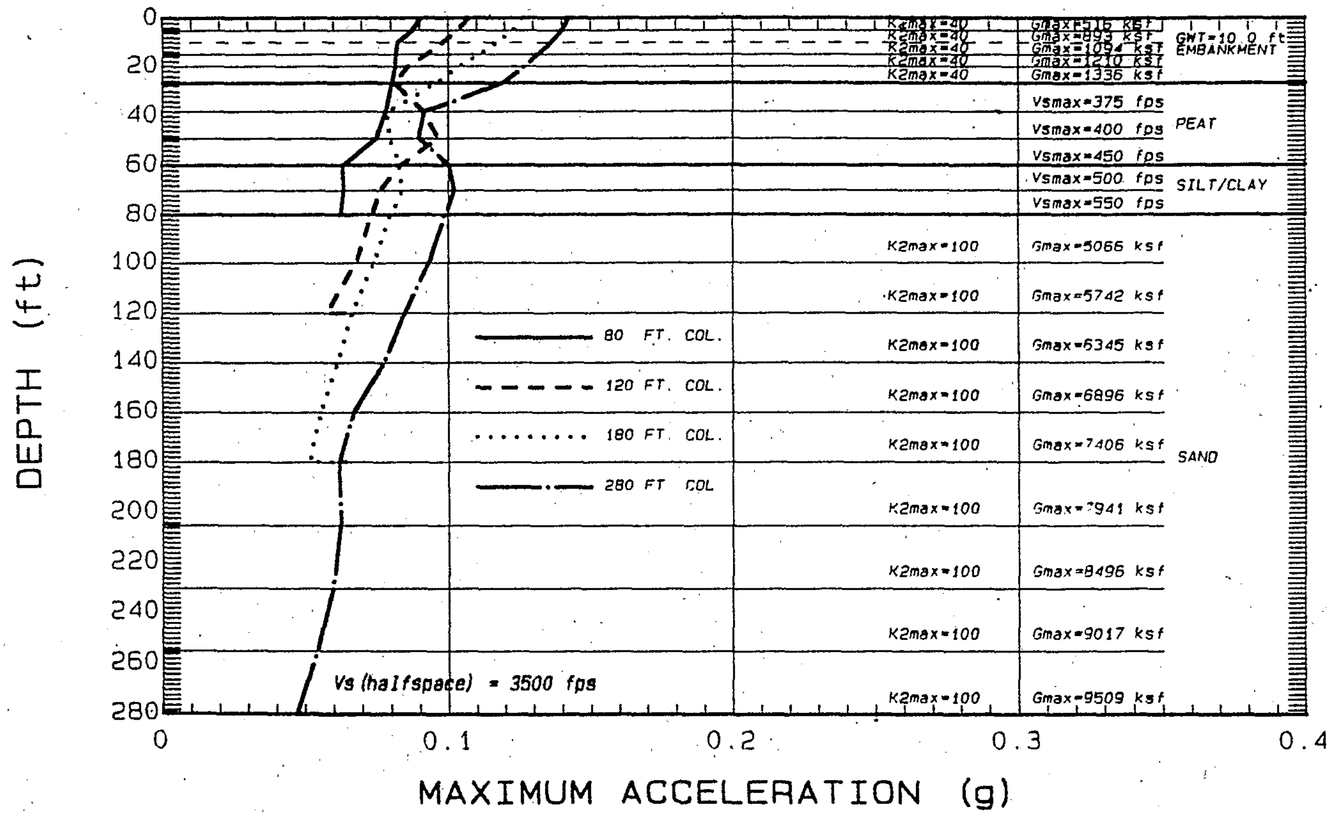


Figure 6-18: Model A Parametric Study - Effect of Varying Model Height. Vs_{max}(halfspace) = 3500 fps.

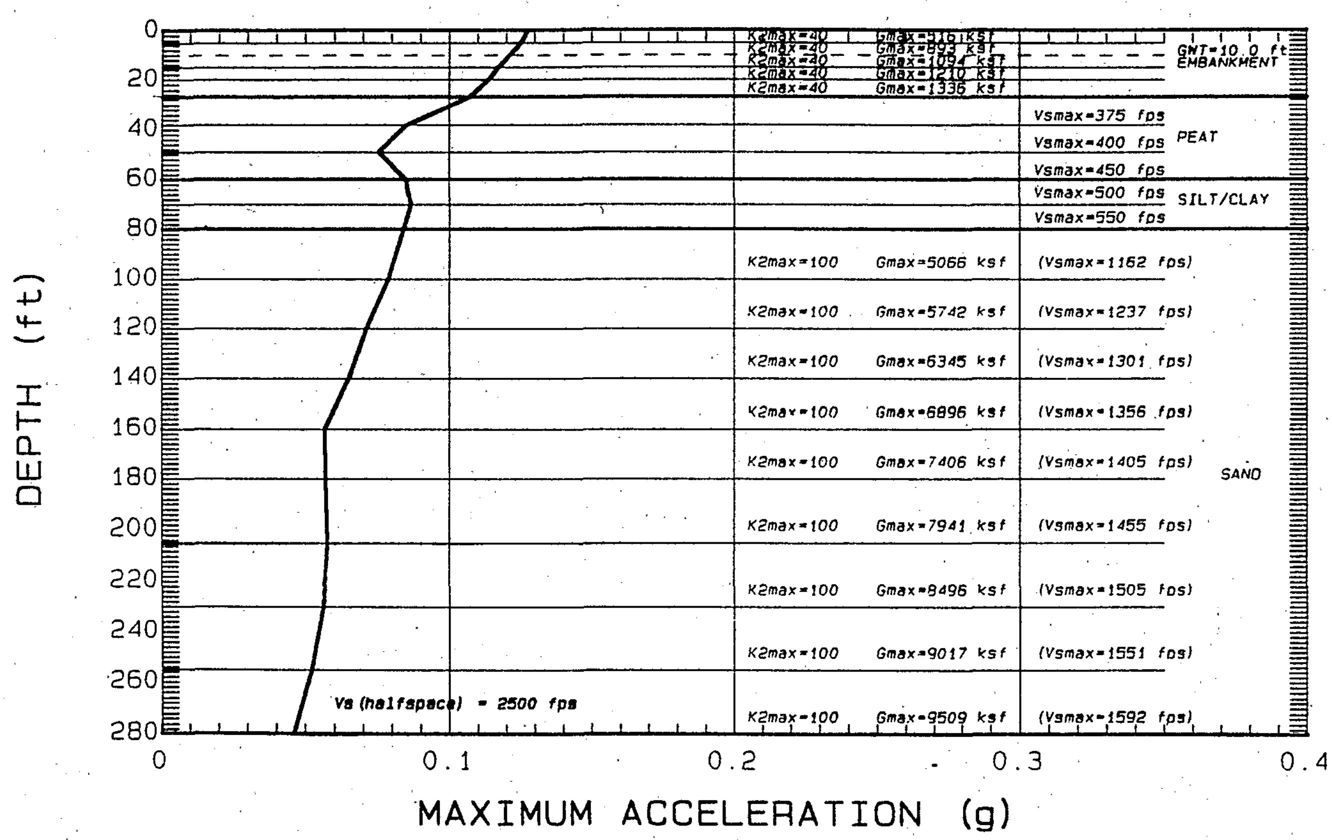


Figure 6-19: Model A Parametric Study - Effect of Varying Deep Soil Types. Sand Below 80 feet.

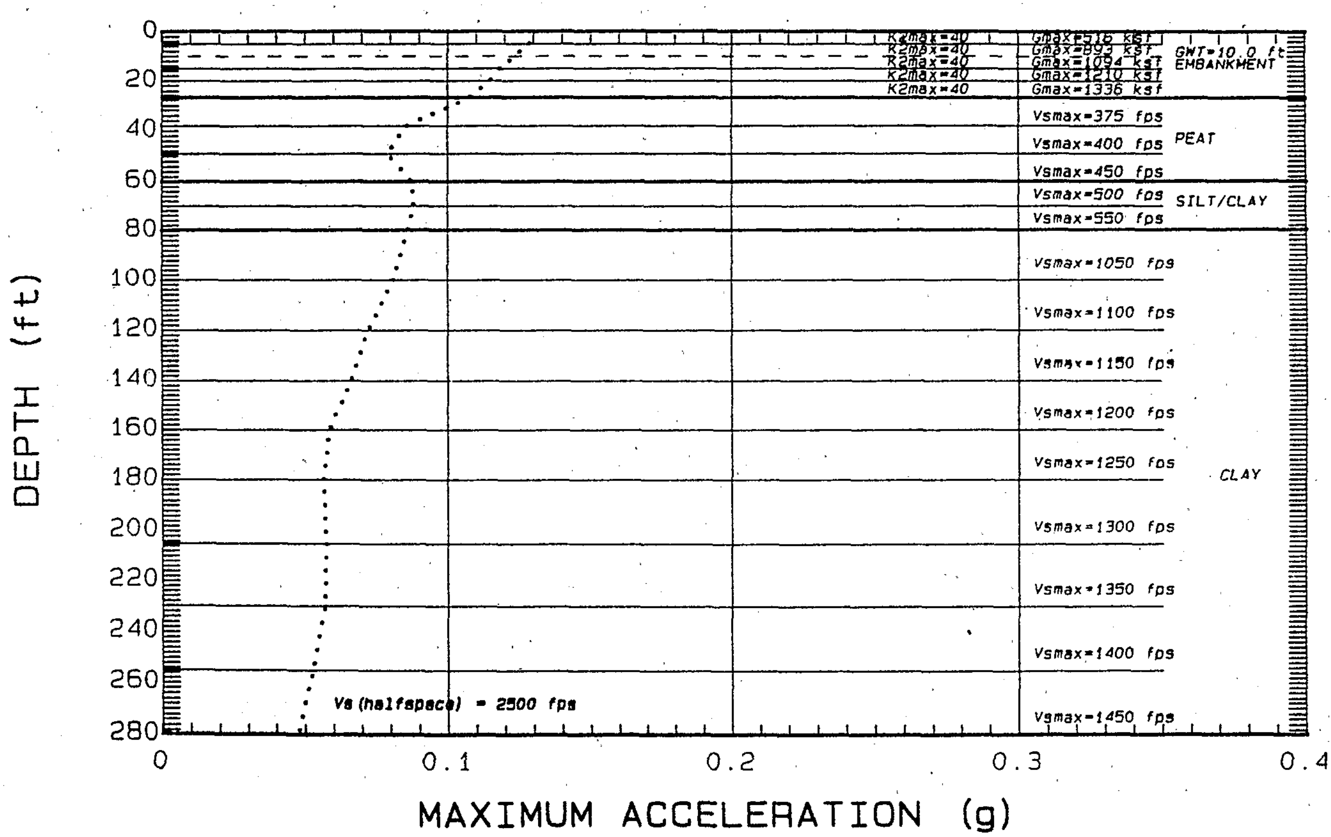


Figure 6-20: Model A Parametric Study - Effect of Varying Deep Soil Types. Clay Below 80 feet.

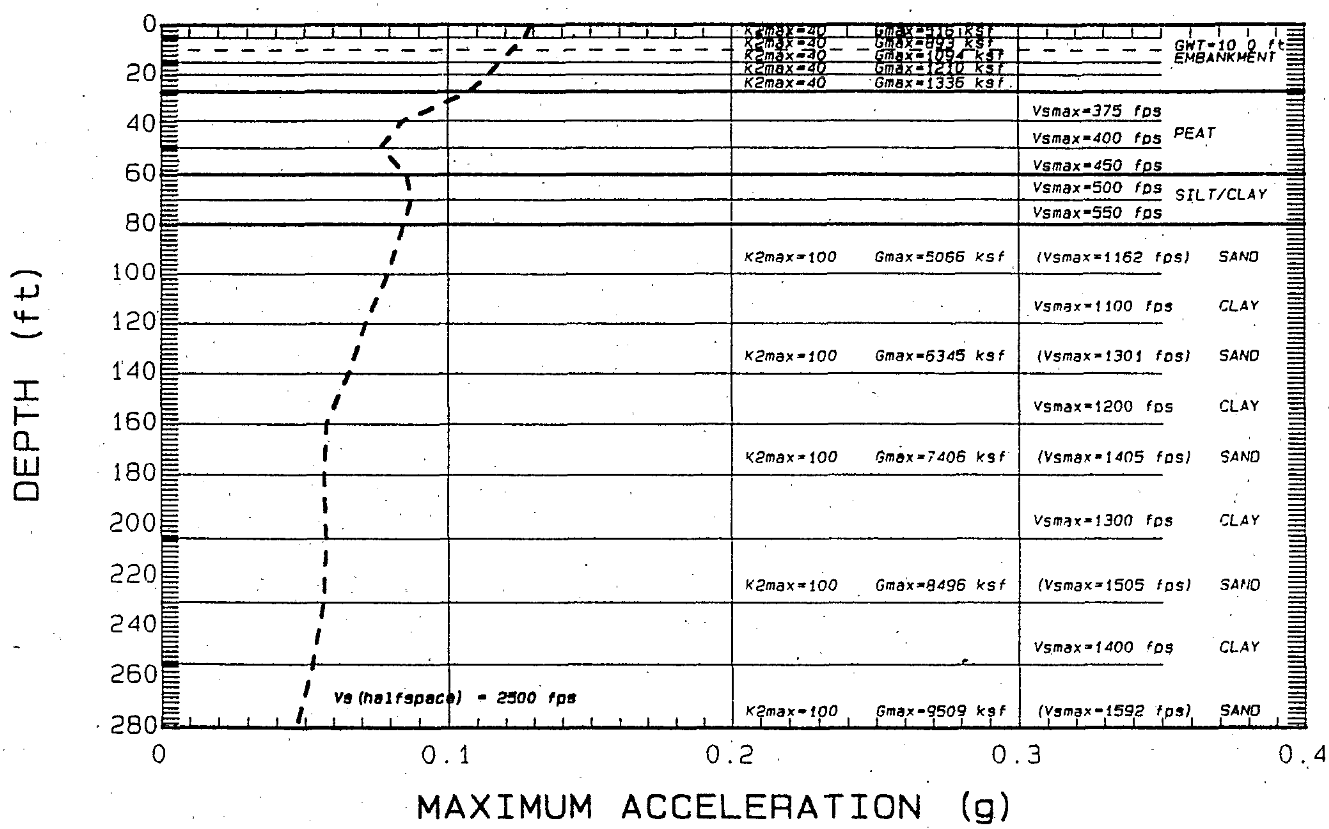


Figure 6-21: **Model A Parametric Study - Effect of Varying Deep Soil Types. Alternating layers of Sand and Clay below 80 feet.**

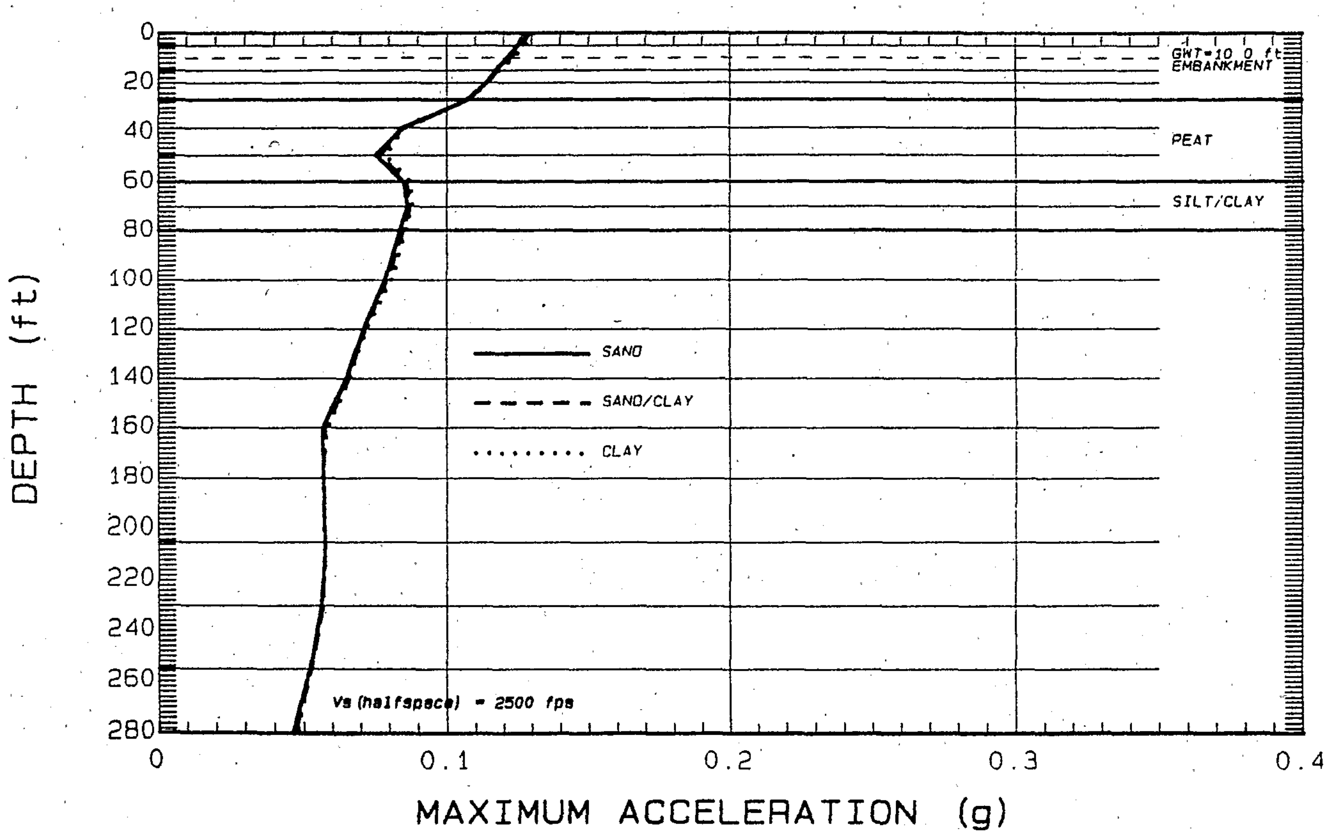


Figure 6-22: Model A Parametric Study - Effect of Varying Deep Soil Types. Summary Plot.

the profile. This result is somewhat surprising and should be further examined in later phases of the investigation.

The results of these analyses indicated a general level of sensitivity for the different parameters that might be adopted for the deeper soils in potential soil profiles. As a result of these analyses, it was decided to adopt the following median values for the response analyses of Models A, B, and C:

- Depth of soil column = 120 feet.
- Halfspace Vs_{max} = 2,500 fps.
- Soil type of deeper soil layers = sand.

In this way, it was hoped that the potential inaccuracies resulting from the unknown characteristics of the soil profiles at depth could be limited as much as possible.

6.6 EFFECT OF PEAT PROPERTIES

The principal purpose for performing the dynamic response analyses was to examine the potential range in either amplification or attenuation of earthquake motions in the peat foundation soils beneath Delta levees. To this end, 81 response analyses were performed using the following permutations:

- 3 response models (Models A, B, and C).
- 3 sets of peat properties.
- 3 earthquake records (Yerba Buena Island, Seed-Idriss, and McCabe).
- 3 earthquake scaling factors (0.10g, 0.15g, and 0.25g).

Plots of acceleration response for all 81 (3x3x3x3) sets of analyses are presented in Appendix F. Three of these plots are represented in Figures 6-23 through 6-25 which show responses for Model A after being loaded with the Yerba Buena Island record scaled to have a peak acceleration of 0.15g. As shown in Figure 6-23, the use of Union Bay properties for the peat layers results in a peak surface motion at the top of the levee of only about 0.11g, a reduction of approximately 30 percent. However, the use of Bay Mud properties for the peat layers results in a peak surface acceleration of 0.21g, or an amplification of approximately 40 percent (see Figure 6-24). The use of fat clay properties for

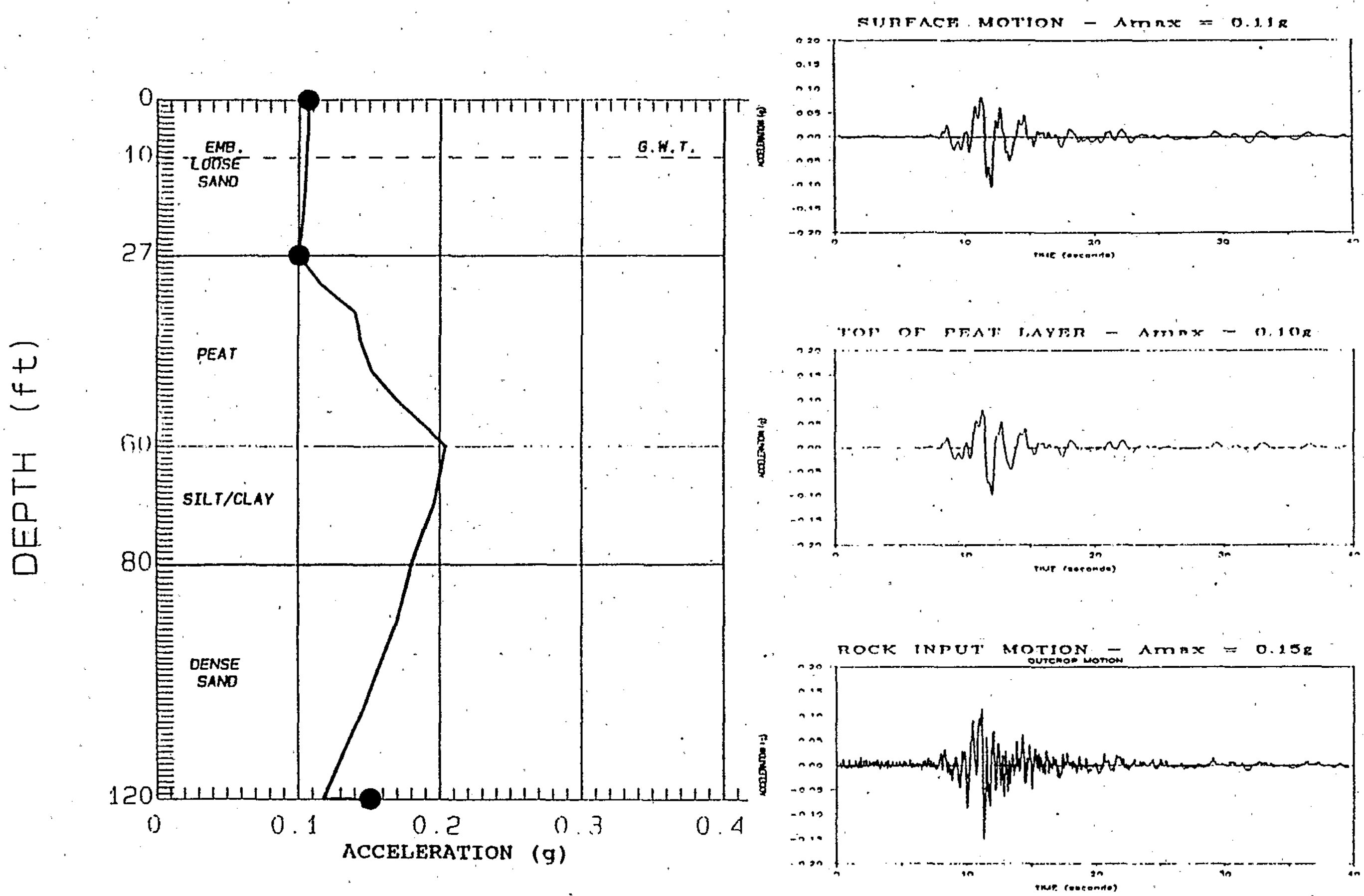


Figure 6-23: Schematic Soil Profile for Model A and Computed Accelerations using Program SHAKE
Peat Modulus = Union Bay Peat (1970)
Peat Damping = Modified Union Bay Peat
Base Motion = Yerba Buena Island (a_{max} = 0.15g - Rock)

C-072389

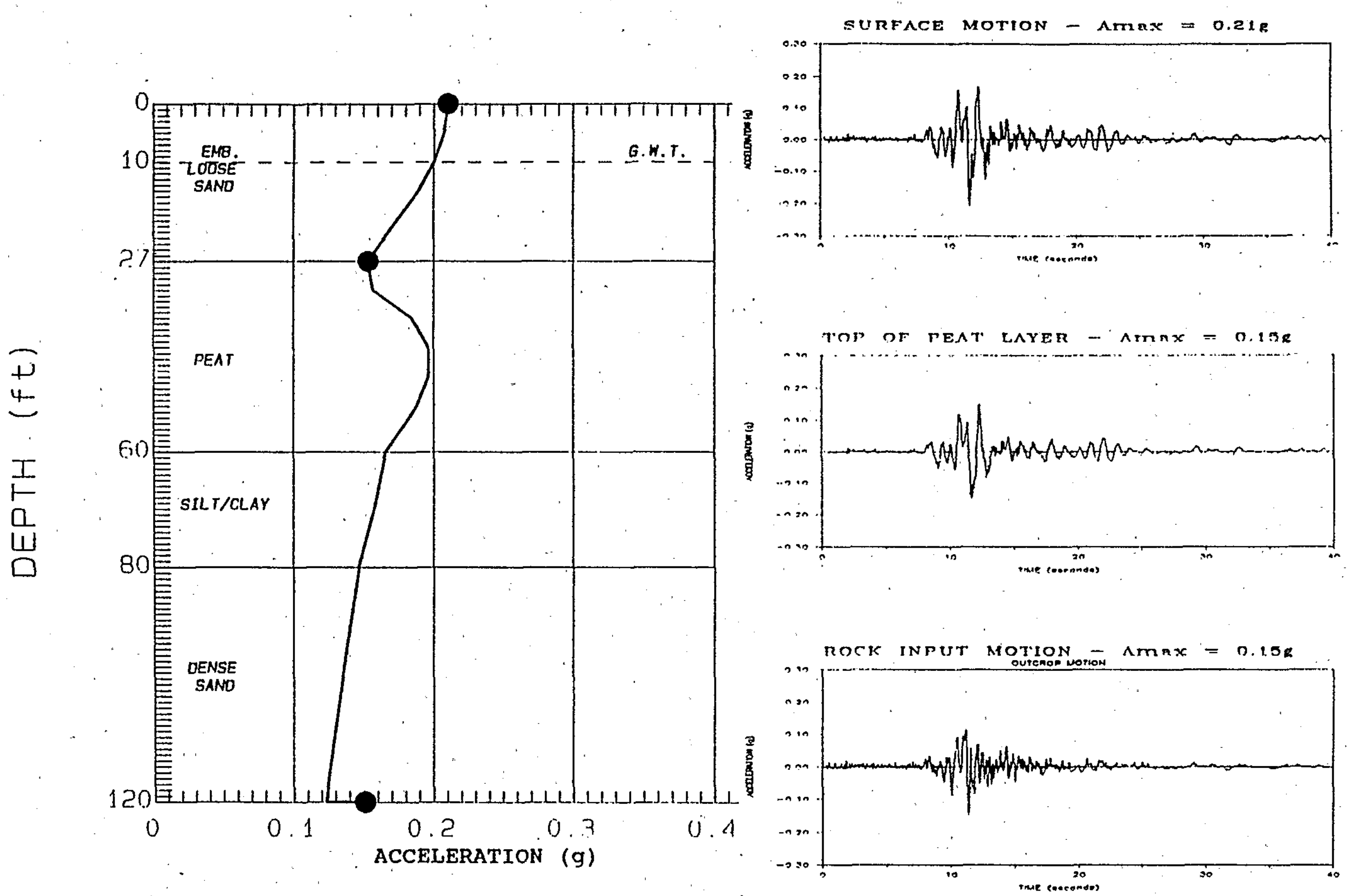


Figure 6-24: Schematic Soil Profile for Model A and Computed Acceleration using Program SHAKE
Peat Modulus = Average Young Bay Mud (1988)
Peat Damping = Clay (1970)
Base Motion = Yerba Buena Island (a_{max} = 0.15g - Rock)

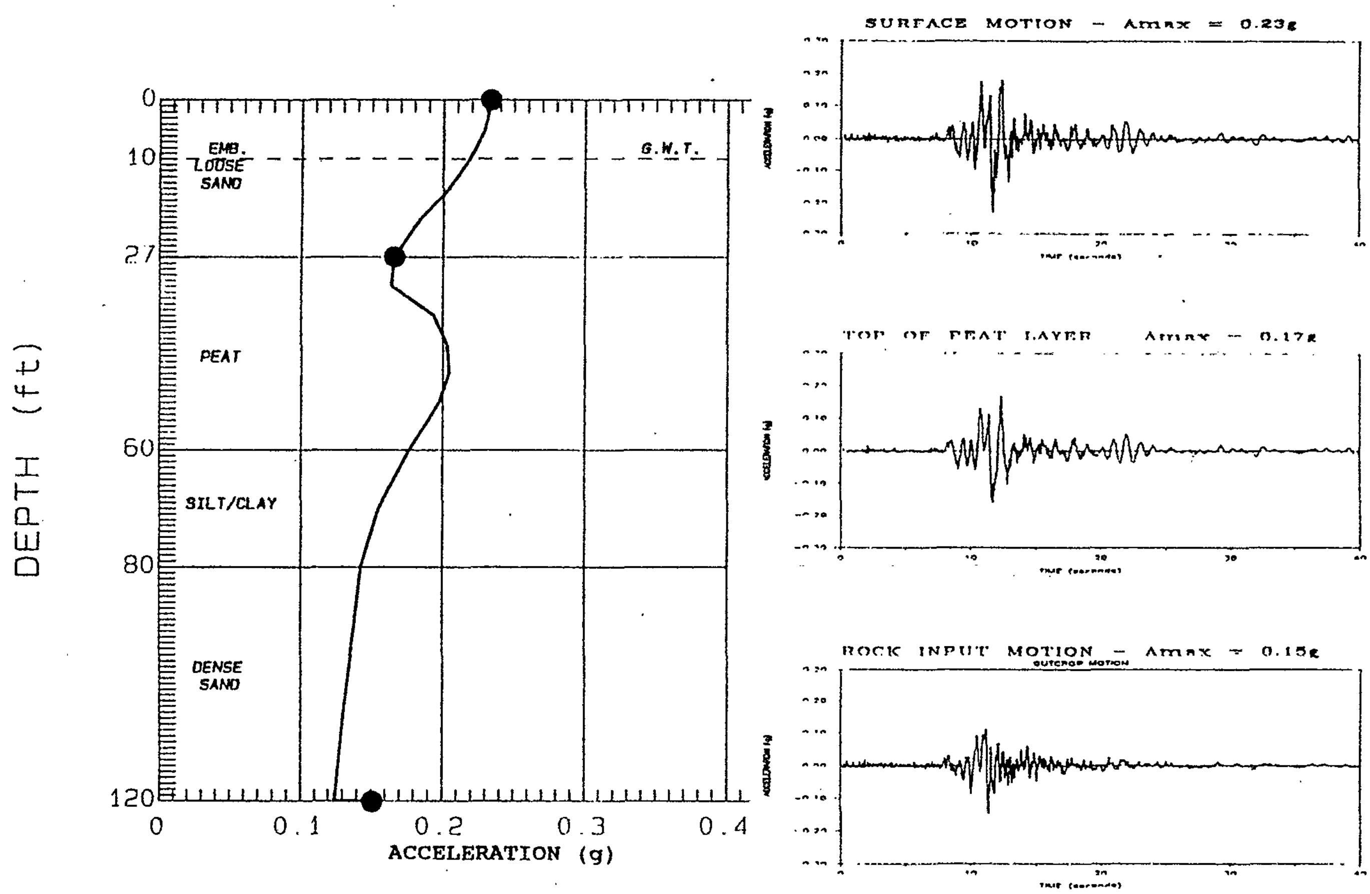


Figure 6-25: Schematic Soil Profile for Model A and
Computed Accelerations Using Program SHAKE
Peat Modulus = Fat Clay (1991)
Peat Damping = Fat Clay (1991)
Base Motion = Yerba Buena Island (a_{max} = 0.15g - Rock)

the peat layers result in an even higher amplification of approximately 55 percent (see Figure 6-25).

The results of the 81 response analyses are summarized graphically in Figures 6-26 through 6-28, and in tabular form in Tables 6-3 through 6-5. The use of the modified Union Bay properties in the peat layers results in amplification ratios between 0.4 and 1.3, but commonly about 0.5. The use of the fat clay properties results in amplification factors between 1.4 and 2.8, but commonly about 2. With such a range in potential ground motion amplification, most of the other unknowns relating to the future performance of Delta levees during earthquakes appear to have lesser significance.

With the large range in potential amplification through the soft peaty soils, it may be necessary to perform extensive field and laboratory tests and/or document the amplification characteristics of peaty soils during actual earthquakes before reliable assessments concerning the seismic stability of Delta levees can be made.

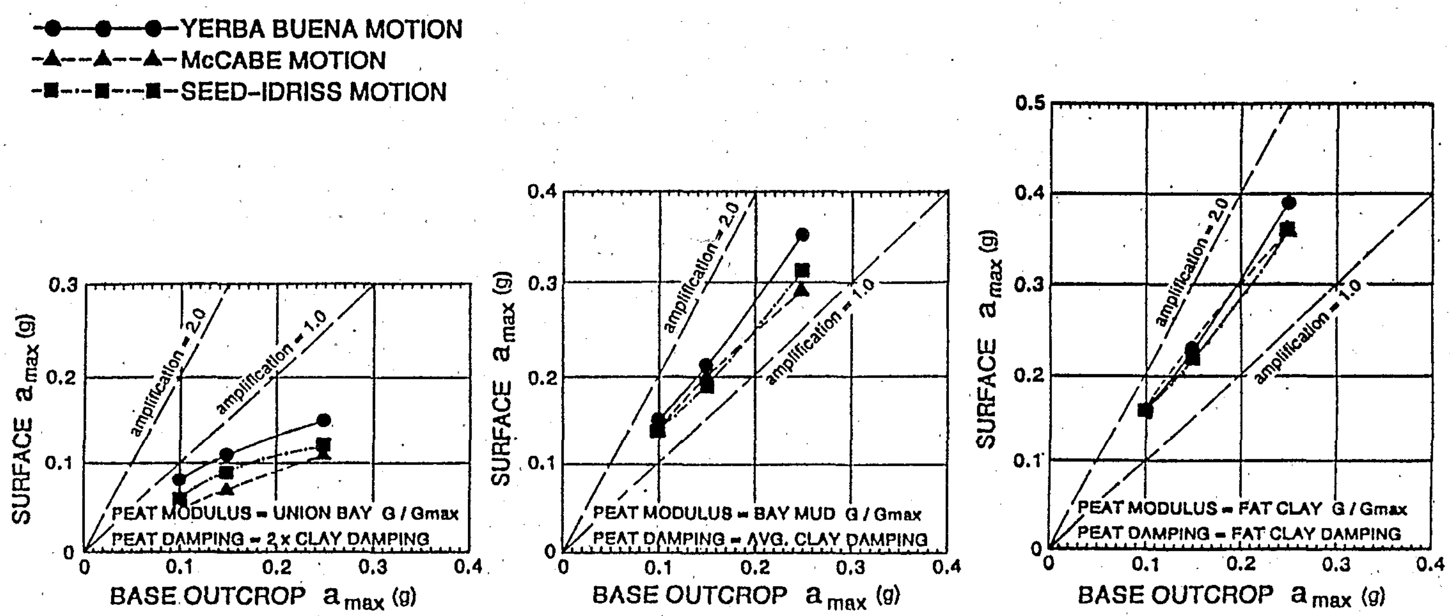


Figure 6-26: Effect of Peat Dynamic Properties on Peak Acceleration Amplification through Profile A

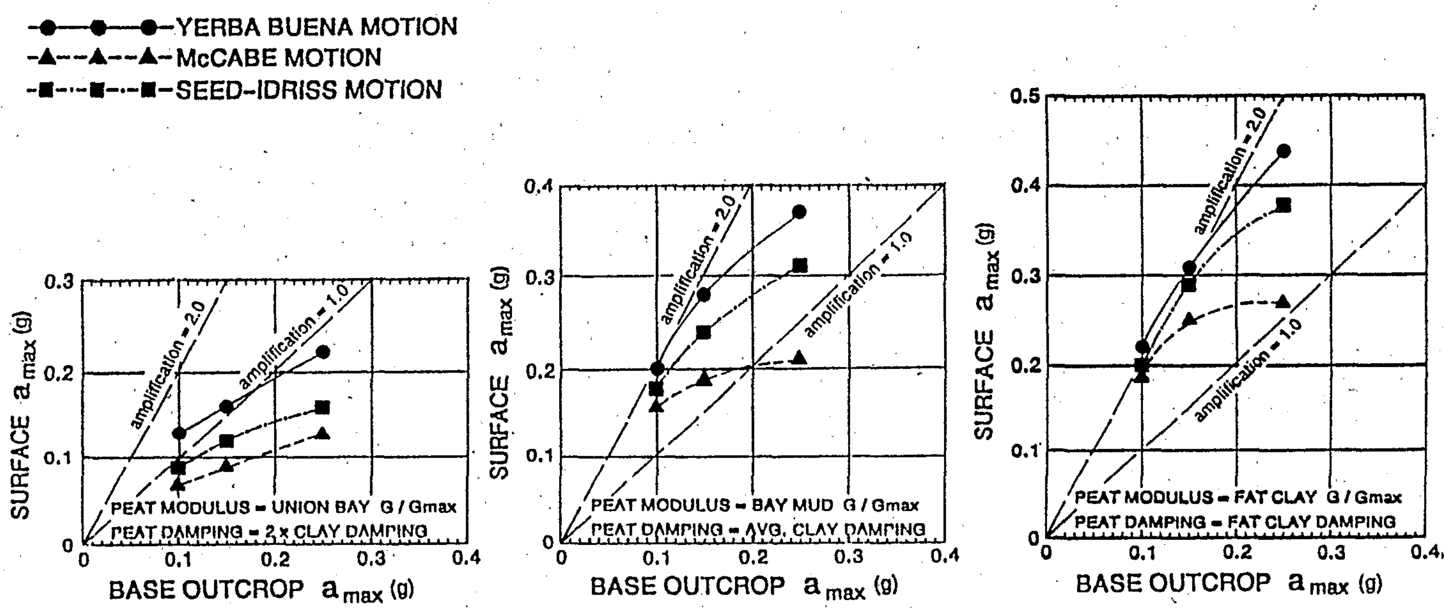


Figure 6-27: Effect of Peat Dynamic Properties on Peak Acceleration Amplification through Profile B

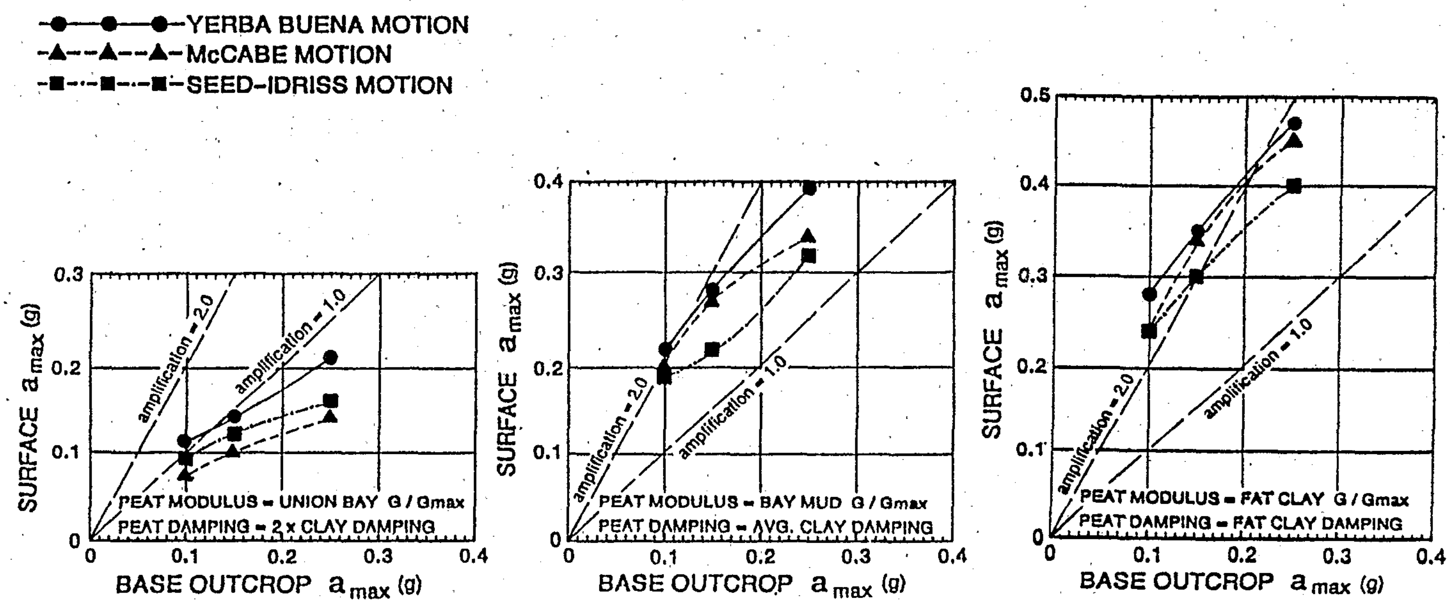


Figure 6-28: Effect of Peat Dynamic Properties on Peak Acceleration Amplification through Profile C

Table 6-3: Results of SHAKE Analysis for Model A

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = UNION BAY PEAT (1970) / PEAT DAMPING = MODIFIED UNION BAY PEAT

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	SEED-IDRISS	MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.08	0.06	0.05	0.5 - 0.8
0.15g	0.11	0.09	0.07	0.5 - 0.7
0.25g	0.15	0.12	0.11	0.4 - 0.6
AMPLIFICATION RANGE (SURFACE/BASE)	0.6 - 0.8	0.5 - 0.6	0.4 - 0.5	

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = AVERAGE YOUNG BAY MUD (1988) / PEAT DAMPING = CLAY (1970)

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	SEED-IDRISS	MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.15	0.14	0.14	1.4 - 1.5
0.15g	0.21	0.19	0.20	1.3 - 1.4
0.25g	0.35	0.31	0.29	1.2 - 1.4
AMPLIFICATION RANGE (SURFACE/BASE)	1.4 - 1.5	1.2 - 1.4	1.2 - 1.4	

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = FAT CLAY (1991) / PEAT DAMPING = FAT CLAY (1991)

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	SEED-IDRISS	MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.16	0.16	0.16	1.6
0.15g	0.23	0.22	0.23	1.5 - 1.6
0.25g	0.39	0.36	0.36	1.4 - 1.6
AMPLIFICATION RANGE (SURFACE/BASE)	1.6	1.4 - 1.6	1.4 - 1.6	

Table 6-4: Results of SHAKE Analysis for Model B

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = UNION BAY PEAT (1970) / PEAT DAMPING = MODIFIED UNION BAY PEAT

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	SEED-IDRISS	MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.13	0.09	0.07	0.7 - 1.3
0.15g	0.16	0.12	0.09	0.6 - 1.1
0.25g	0.22	0.16	0.13	0.5 - 0.9
AMPLIFICATION RANGE (SURFACE/BASE)	0.9 - 1.3	0.6 - 0.9	0.5 - 0.7	

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = AVERAGE YOUNG BAY MUD (1988) / PEAT DAMPING = CLAY (1970)

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	SEED-IDRISS	MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.20	0.18	0.16	1.6 - 2.0
0.15g	0.28	0.24	0.19	1.2 - 1.8
0.25g	0.37	0.31	0.21	0.8 - 1.5
AMPLIFICATION RANGE (SURFACE/BASE)	1.5 - 2.0	1.2 - 1.8	0.8 - 1.6	

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = FAT CLAY (1991) / PEAT DAMPING = FAT CLAY (1991)

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	SEED-IDRISS	MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.22	0.20	0.19	1.8 - 2.2
0.15g	0.31	0.29	0.25	1.6 - 2.1
0.25g	0.44	0.38	0.27	1.1 - 1.8
AMPLIFICATION RANGE (SURFACE/BASE)	1.7 - 2.2	1.5 - 2.0	1.1 - 1.8	

Table 6-5: Results of SHAKE Analysis for Model C

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = UNION BAY PEAT (1970) / PEAT DAMPING = MODIFIED UNION BAY PEAT

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	EARTHQUAKE MOTION: SEED-IDRISS	EARTHQUAKE MOTION: MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.11	0.09	0.07	0.7 - 1.1
0.15g	0.14	0.12	0.10	0.6 - 1.0
0.25g	0.21	0.16	0.14	0.6 - 0.8
AMPLIFICATION RANGE (SURFACE/BASE)	0.8 - 1.1	0.6 - 0.9	0.6 - 0.7	

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = AVERAGE YOUNG BAY MUD (1988) / PEAT DAMPING = CLAY (1970)

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	EARTHQUAKE MOTION: SEED-IDRISS	EARTHQUAKE MOTION: MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.22	0.19	0.20	1.9 - 2.2
0.15g	0.28	0.22	0.27	1.5 - 1.9
0.25g	0.39	0.32	0.34	1.3 - 1.6
AMPLIFICATION RANGE (SURFACE/BASE)	1.6 - 2.2	1.3 - 1.9	1.3 - 2.0	

COMPUTED MAXIMUM SURFACE ACCELERATION (g)
PEAT MODULUS = FAT CLAY (1991) / PEAT DAMPING = FAT CLAY (1991)

Amax, BASE INPUT MOTION	EARTHQUAKE MOTION: YERBA BUENA ISLAND	EARTHQUAKE MOTION: SEED-IDRISS	EARTHQUAKE MOTION: MCCABE	AMPLIFICATION RANGE (SURFACE/BASE)
0.10g	0.28	0.24	0.24	2.4 - 2.8
0.15g	0.35	0.30	0.34	2.0 - 2.3
0.25g	0.47	0.40	0.45	1.6 - 1.9
AMPLIFICATION RANGE (SURFACE/BASE)	1.9 - 2.8	1.6 - 2.4	1.8 - 2.4	

7. SEISMIC STABILITY

7.0 GENERAL

Levee failure is defined as sufficient levee distress as to result in inundation of the protected area, usually a Delta island. The slope stability of levees and other embankments following earthquakes is generally evaluated by examining two potential modes of distress:

1. The soils comprising the embankment and/or foundation may liquefy and/or otherwise lose significant strength, resulting in slope failure and/or large deformations.

2. Even for soils which do not liquefy, the inertial forces resulting from the earthquake shaking can cause the soil to yield during significant pulses of motion, leading to incremental amounts of deformation. If there are sufficient numbers of such pulses, the accumulated incremental movements developed during the pulses can be large enough to be unacceptable.

Both of the above modes of distress are considered possible for the levees in the Sacramento-San Joaquin Delta. The levees and their foundations often contain liquefiable sand and/or silt, and the soft organic foundation is susceptible to earthquake-induced deformations. Thus, the specific concerns related to earthquake shaking are:

- Liquefaction of levee fill.

- Liquefaction of levee foundation.

- Incremental deformation of levee and foundation due to inertial forces.

These specific modes of levee distress lead to deformations (e.g., slumping and spreading) of the levee which may result in levee failure and island inundation by either:

- Levee cracking leading to internal erosion or piping.

or

- Loss of freeboard and levee overtopping.

7.1 LIQUEFACTION POTENTIAL OF LEVEES

The general level of data available to characterize the liquefaction potential of Delta levees and their foundations is very limited both in quantity and quality. As noted in the 1987 U. S. Army Corps of Engineers studies, liquefiable soils commonly exist throughout the Delta (see Figure 3-7). However, the precise level of shaking required to trigger extensive liquefaction and the degree of continuity cannot be reliably assessed for most areas of the Delta with the information now available.

Some insight, however, can be obtained by examining Standard Penetration Test (SPT) data obtained within the sandy south levee on Sherman Island. This information was obtained in the recent studies of this levee by Roger Foote and Associates. As shown in Figure 7-1, the sandy and silty levee materials have corrected SPT blowcounts, $(N_1)_{60}$, ranging generally between 0 and 30 blows per foot. However, below the water level, the penetration resistance is typically about 5 blows per foot, an extremely low value for cohesionless soils. Based on this information and the data shown in Figures 2-13 and 2-14 and other references (e.g., Converse and Associates, 1981), it seems reasonable to assume that silty and/or sandy levees in the Delta commonly have corrected blowcounts between 5 and 10 blows per foot and have fines contents ranging up to about 35 percent or more. Given this assumption, the correlation developed by Seed, et al. (1985), would indicate that the cyclic stress ratio required to trigger liquefaction in many levees during a Magnitude 7.5 earthquake would generally be expected to be between 0.13 and 0.18 (see Figure 7-2).

In order to determine what level of shaking is required to trigger extensive liquefaction, it is necessary to estimate the level of dynamic stress that might be induced in the sandy levee. This can most simply be estimated using the following equation from Seed, et al. (1985):

$$\tau_{AVG} = 0.65 \times \sigma_o \times \frac{a_{max}}{g} \times r_d \qquad (7.1)$$

where τ_{AVG} = Earthquake-induced cyclic shear stress.

σ_o = Total overburden pressure at depth.

a_{max} = Maximum acceleration at levee surface.

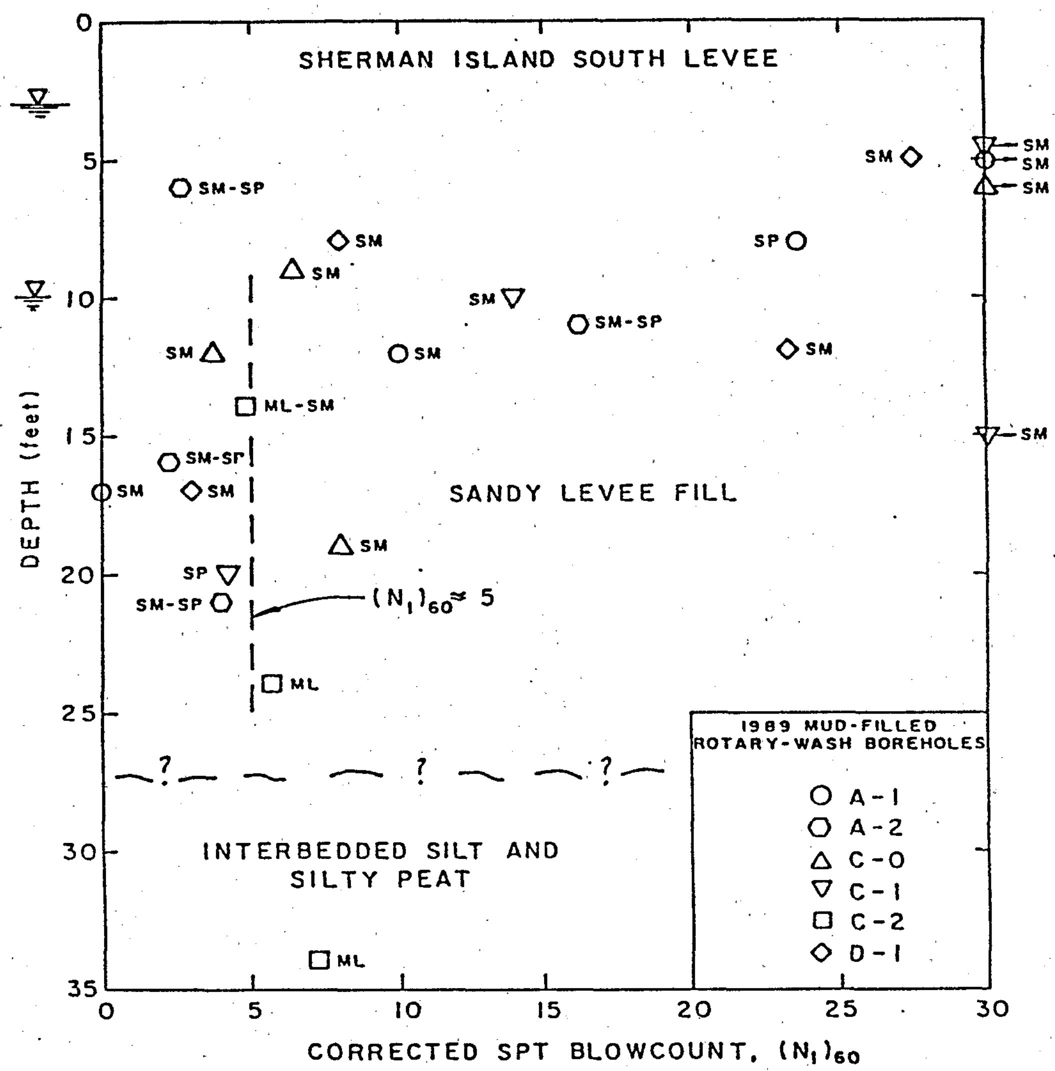


Figure 7-1: Corrected Blowcount Values, Sherman Island South Levee

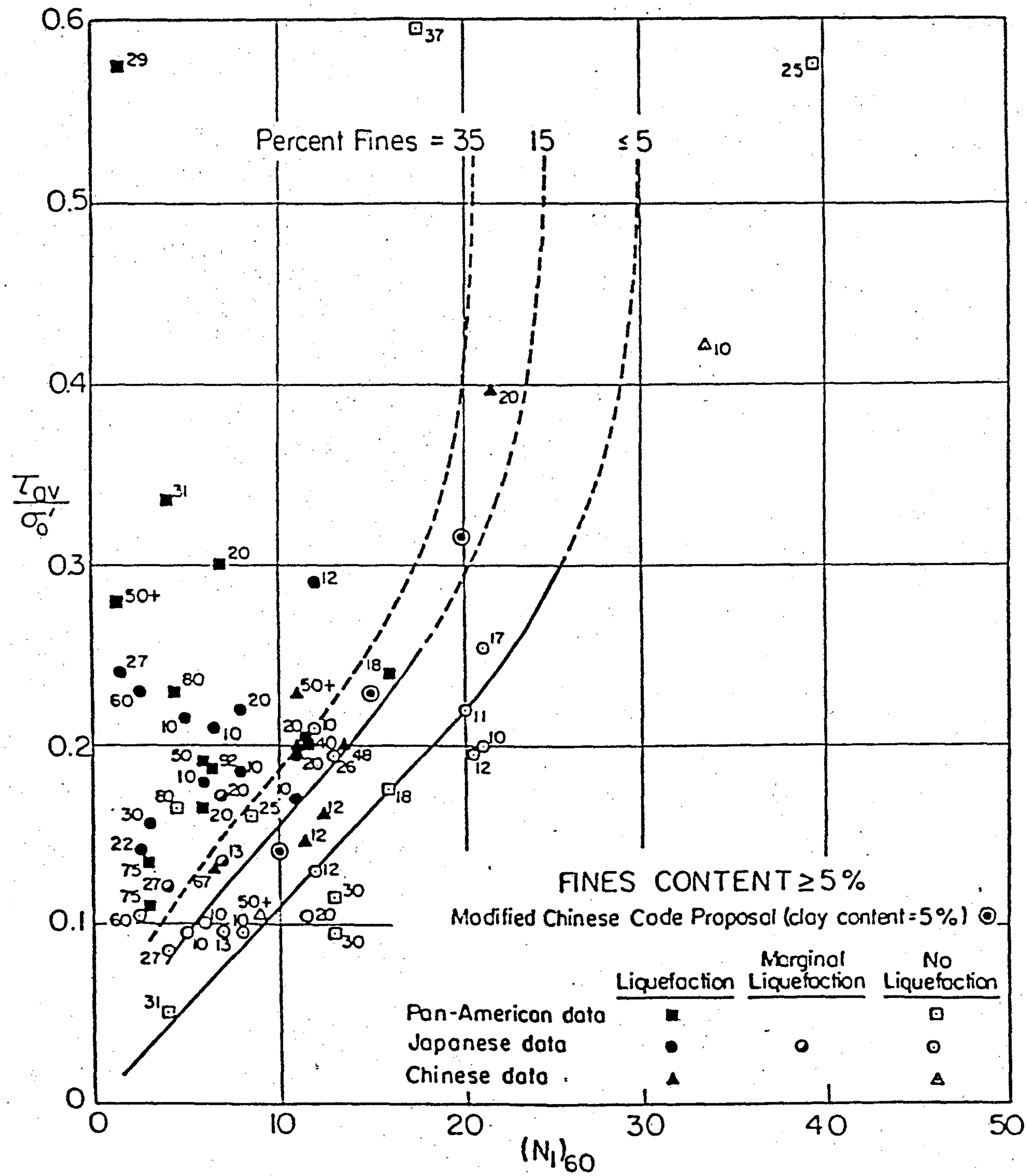


Figure 7-2: Relationship Between Stress Ratio Causing Liquefaction and N_1-Values for Silty Sands for M = 7.5 Earthquakes. (from Seed et al. 1985).

r_d = A stress reduction factor varying from unity at the ground surface to a value of about 0.85 at 40 feet.

To estimate the liquefaction resistance for reasonably level ground at moderate overburden pressures, Seed and Harder (1990) suggest the following equation:

$$\tau_L = (\tau_L/\sigma_o')_{M=7.5} \times \sigma_o' \qquad (7.2)$$

where τ_L = Cyclic load resistance to liquefaction (normalized to M=7.5).

$(\tau_L/\sigma_o')_{M=7.5}$ = Cyclic resistance ratio from SPT correlation.

σ_o' = Effective overburden pressure at depth.

Figure 7-3 presents the results of calculations assuming a cyclic resistance ratio of 0.15 for two different levee locations. The first location, typical of a location along a levee crown, has a depth to water of 10 feet. The second location, typical of many flat sandy berms, has a depth to water of 3 feet. Liquefaction would be expected when the average earthquake-induced cyclic stress, τ_{AVG}, exceeded the cyclic load resistance, τ_L. These results indicate that liquefaction would commonly result in cohesionless levee materials when peak ground accelerations at the levee surface reach approximately 0.15g to 0.2g.

7.2 LIQUEFACTION POTENTIAL OF LEVEE FOUNDATIONS

Several sources have theorized that extensive layers of sand lie beneath the peat in many areas of the Delta and that these sand layers are liquefiable. Data regarding the liquefaction potential of foundation sands are even more limited than for the levees themselves. As shown in Figure 2-14, there appears to be a thin sandy layer with low blowcounts beneath the levee berm. However, the penetration resistance values beneath the levee itself are relatively high, thus indicating a possible lack of continuity. In Figure 2-13, a sandy layer exists beneath the peaty organic soil, but the blowcounts are generally high, thus indicating a relatively low potential for liquefaction in this material. In Figure 2-12, there is no sand layer in the

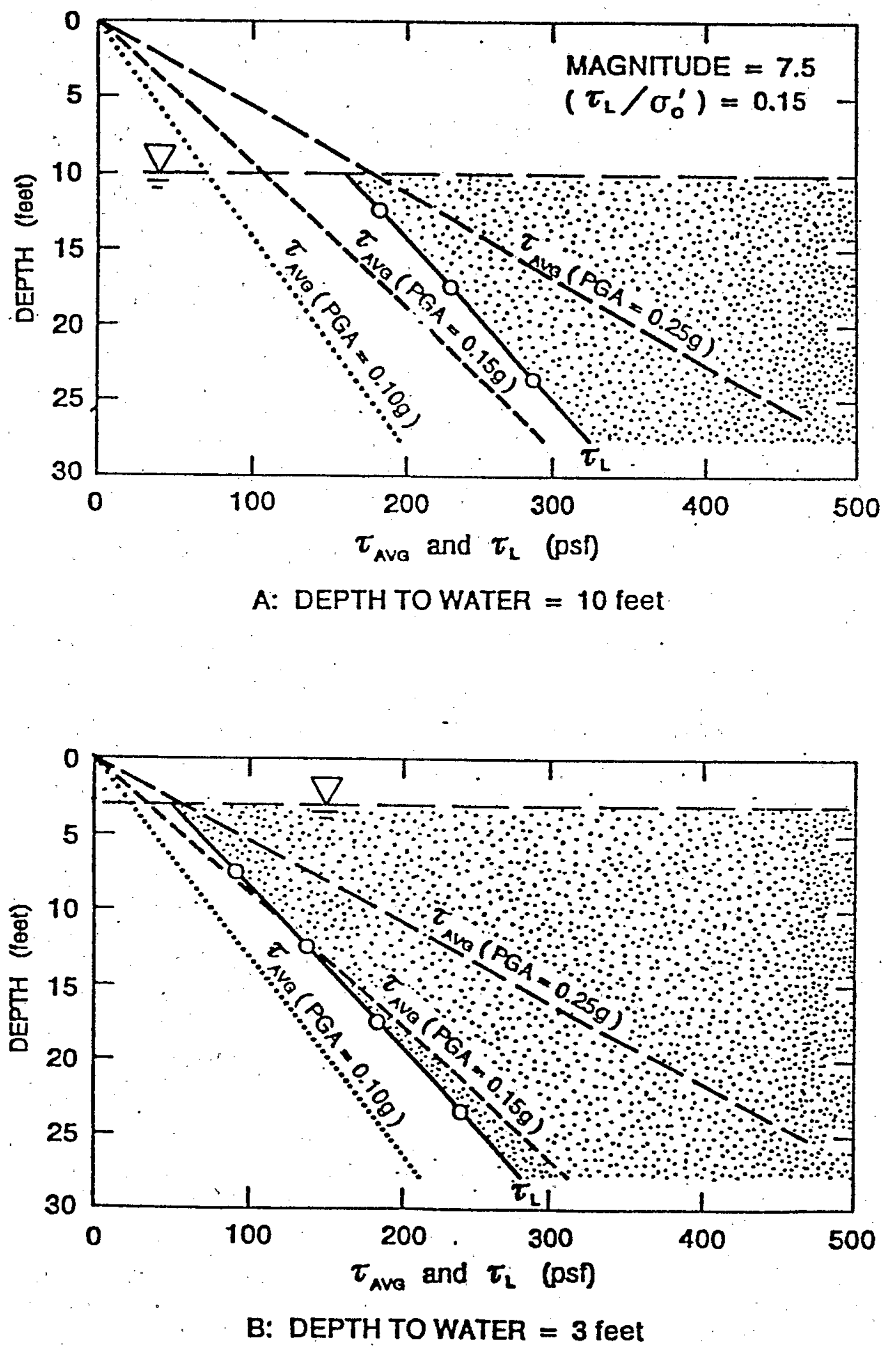


Figure 7-3: Liquefaction Potential in Cohesionless Levees

foundation beneath the peaty soil until a depth of about 70 feet is reached, and this sand is quite dense. Based on this very limited data, it would seem likely that the potential for levee failure from liquefaction of foundation sands may be less serious than potential liquefaction of the levees themselves.

It should also be noted that the potential amplification characteristics of the peaty soils generally do not influence the triggering of liquefaction of sandy soils lying beneath them. Therefore, scenarios involving potential amplification factors of 2 to 3 for earthquake motions through the peat do not apply for calculations involving liquefaction potential of foundation sands.

There is also no reliable data concerning the potential for significant strength losses in soft clayey and peaty soils during earthquake shaking. Although clayey fills generally perform well during earthquakes (Seed, et al., 1976), it is not known if this good performance is typical for the soft clayey and peaty soils common in the levee foundation. The only available case history of an earthquake-induced failure of an embankment founded on peaty soil is the railroad embankment failure in the Suisun Marsh during the 1906 San Francisco Earthquake (Magnitude 8+, see Section 5.64). Outcrops of bedrock in this area were estimated to have experienced a peak acceleration of approximately 0.18g.

7.3 EFFECT OF LIQUEFACTION ON SLOPE STABILITY

7.30 General

Previous sections have discussed the potential for liquefaction development in either the levee or its foundation. However, the development of limited extents of liquefaction do not necessarily lead to significant deformations. Even the development of a continuous layer of liquefied material may not lead to levee failure if there exists sufficient residual shear strength left in the liquefied soil to resist the driving forces induced by gravity.

In order to examine whether the development of extensive liquefaction in either the levee or its foundation would result in levee failure, several post-earthquake slope stability analyses were performed. These analyses assumed a range of possible residual shear strengths for liquefied zones in either the levee or foundation and then computed factors of safety against sliding.

7.31 Slope Stability Model

A generic levee model was developed for the stability analyses. This model is depicted in Figure 7-4 and represents a site that has an organic layer which is approximately 25 feet thick beneath the levee. The levee is approximately 20 feet high and has a landside slope of approximately 5:1. Underlying the organic soil layer are layers of sand, silts, clays. All of these layers rest upon a dense sand layer. These conditions are considered to be common throughout the Delta, although possibly not continuous for any particular levee reach.

The material properties for non-liquefied soils were developed by using drained strengths for sandy soils and undrained shear strengths for clayey and/or peaty soils. Shear strengths were chosen based on values used for other studies performed by the Department (e.g., Driller, 1990; Montezuma Slough, 1986) and by others (e.g., Dames and Moore, 1986; Roger Foote and Associates, 1991; Ake, et al., 1991). These studies included field and laboratory test programs, as well as stability evaluations. The specific properties adopted were as follows:

Cohesionless Levee Fill - An effective friction angle of 30 degrees and no cohesion is used to represent a non-liquefied sand embankment. It is also reasonable to use this strength to model a cohesive levee which has cracked (Chirapuntu and Duncan, 1977).

Organic Soil - Undrained shear strengths were selected for the organic soils. The organic soil layer was divided into four zones, each representing a particular degree of consolidation and related strength. Beneath the embankment this organic layer is subjected to the greatest load. Consequently, over time it has consolidated and gained strength. As the levee slope tapers toward the landside the consolidation stresses are less, as are the corresponding strengths. Beyond the toe, the soil is relatively normally consolidated and has the lowest strength. These values are in good agreement with strengths obtained using a field vane shear device. Since it is not known if the strength of the peaty, organic layer is affected by earthquake shaking, no modification of the static strengths were made.

Soil Layers Below the Organic Layer - The sand and silt/clay layers below the organic soil layer were assumed to be stronger than the organic soil layer. Appropriate drained and undrained strengths were assigned accordingly.

	MATERIAL DESCRIPTION	DENSITY (pcf)	COHESION (psf)	FRICTION ANGLE (degrees)
1	WATER	62.4	—	—
2	MOIST LEVEE FILL	115	0	30
3	SATURATED LEVEE FILL	130	0	30*
4	DRY ORGANIC CRUST	60	500	0
5	ORGANIC SOIL 1	70	225	0
6	ORGANIC SOIL 2	80	450	0
7	ORGANIC SOIL 3	90	650	0
8	ORGANIC SOIL 4	95	850	0
9	ORGANIC SOIL 5	90	650	0
10	RIVER SAND	125	0	35
11	FOUNDATION SAND	125	0	35*
12	FOUNDATION SILT/CLAY	125	1000	0
13	DENSE SAND	130	0	35

*SOIL ALSO ANALYZED WITH POST-LIQUEFACTION RESIDUAL SHEAR STRENGTH, S_r

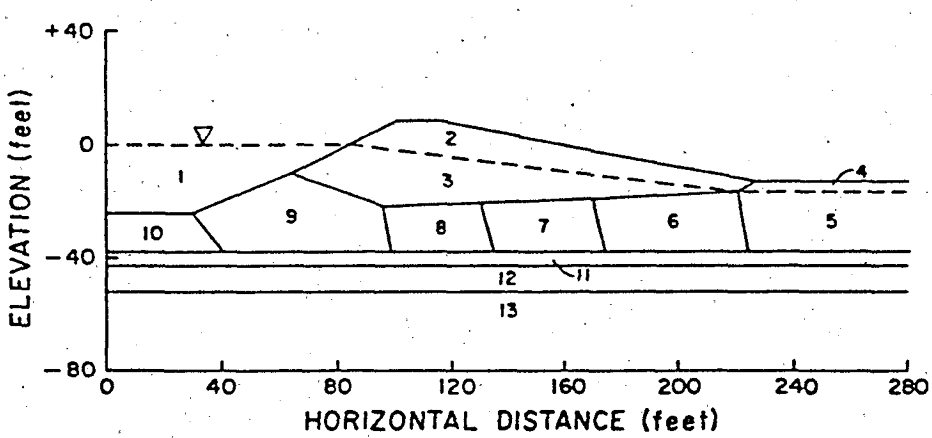


Figure 7-4: Slope Stability Model

7.32 Method of Analysis

The calculations were performed using computer program PCSLOPE (Geo-Slope, International, Ltd.). The method analysis employed in the computations was the Modified Bishop Method. Using the above properties for conditions of no liquefaction, the static factor of safety was computed to be 1.24. The depth of the critical sliding surface is near the bottom of the organic soil layer as shown in Figure 7-5. This result is comparable to the results from other studies of levee stability in the Delta and was considered a good check on the properties assumed.

7.33 Residual Shear Strength of Liquefied Zones

The effects of both levee liquefaction and foundation liquefaction were examined in separate analyses by assuming different zones in the model had liquefied. Within the embankment, the fill beneath the phreatic line was assumed to be liquefied and assigned a range of possible residual shear strengths. For the foundation, a 5-foot horizonal layer of sand immediately below the organic soil was assumed to have liquefied and to have only residual shear strength values.

7.34 Post-Earthquake Stability Analysis Results

Figure 7-6 presents computed factors of safety as a function of residual shear strength for conditions of levee liquefaction and for foundation liquefaction. Also shown on this figure is the approximate equivalent clean sand SPT blowcount, $[(N_1)_{60}]_{cs}$, that would correspond to the residual shear strength value. The SPT value was obtained from the correlation suggested by Seed and Harder (1990).

The results of the stability computations indicate that the saturated portion of the levee must develop a residual shear strength of approximately 250 psf to develop a factor of safety of unity. This corresponds to an equivalent clean sand SPT blowcount, $[(N_1)_{60}]_{cs}$, of approximately 11 blows per foot. Because available data suggests that Delta levees contain sandy soils with equivalent $[(N_1)_{60}]_{cs}$ values commonly less than 10 (see Figures 7-1, 2-13, and 2-14), the analyses indicate that the development of extensive liquefaction would lead to large deformations and possible failure in many levees in the Delta. This is a conclusion reached by many other previous studies (see Chapter 3).

	MATERIAL DESCRIPTION	DENSITY (pcf)	COHESION (psf)	FRICTION ANGLE (degrees)
1	WATER	62.4	—	—
2	MOIST LEVEE FILL	115	0	30
3	SATURATED LEVEE FILL	130	0	30*
4	DRY ORGANIC CRUST	60	500	0
5	ORGANIC SOIL 1	70	225	0
6	ORGANIC SOIL 2	80	450	0
7	ORGANIC SOIL 3	90	650	0
8	ORGANIC SOIL 4	95	850	0
9	ORGANIC SOIL 5	90	650	0
10	RIVER SAND	125	0	35
11	FOUNDATION SAND	125	0	35*
12	FOUNDATION SILT/CLAY	125	1000	0
13	DENSE SAND	130	0	35

*SOIL ALSO ANALYZED WITH POST-LIQUEFACTION RESIDUAL SHEAR STRENGTH, S_r

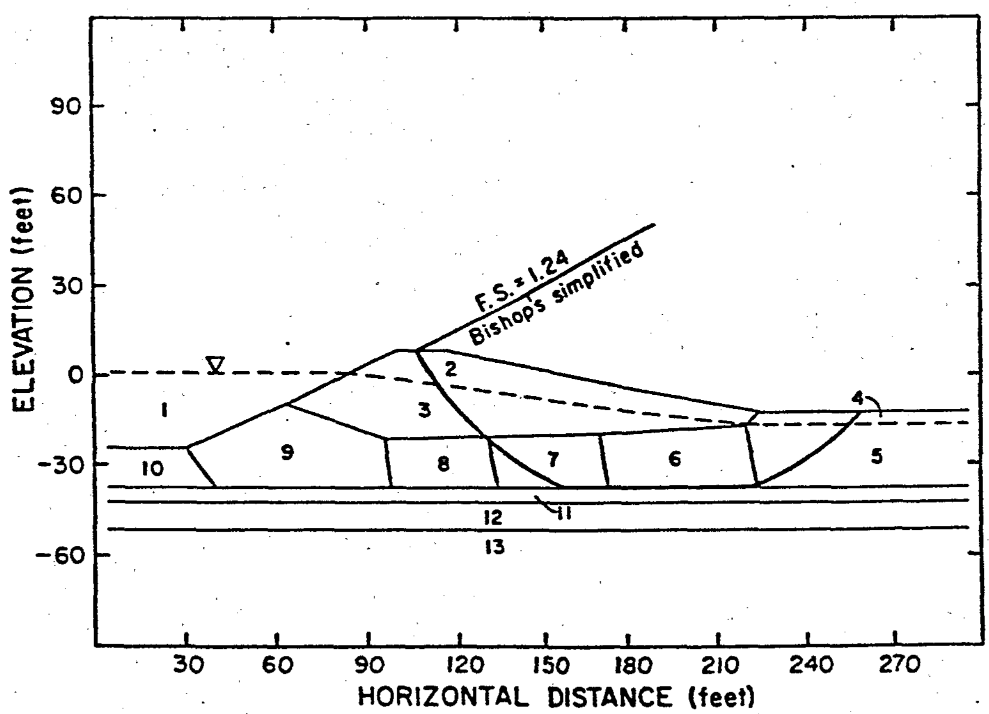


Figure 7-5: Location of Critical Sliding Surface for Static Loading Conditions.

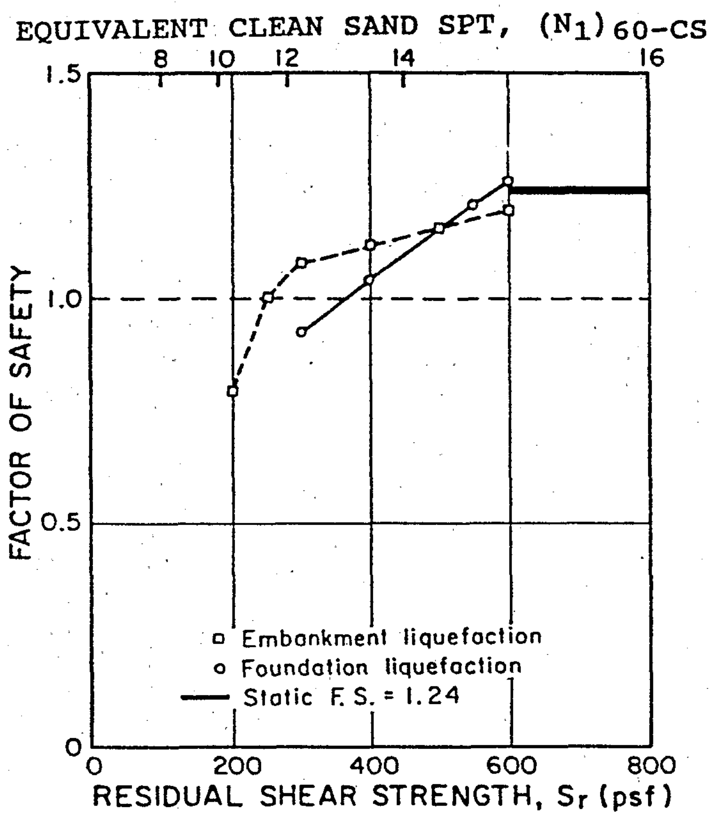


Figure 7-6: Residual Strength and $(N_1)_{60-cs}$ vs Factor of Safety for Post-Earthquake Slope Stability Analysis

Figure 7-6 also shows that the liquefied foundation sand layer must have a residual shear strength of approximately 350 psf before a factor of safety of unity is reached. This corresponds to an equivalent clean sand SPT blowcount, $[(N_1)_{60}]_{cs}$, of approximately 13 blows per foot. These results indicate that levees would be expected to undergo significant deformations following the onset of liquefaction where embankments or foundations have continuous soil layers with equivalent clean sand SPT blowcounts less than about 11 to 13. However, the results also suggest that if the equivalent clean sand SPT blowcounts were greater than about 15 blows per foot, then many levees would remain stable, even if liquefaction did develop.

7.4 EARTHQUAKE-INDUCED DEFORMATIONS IN LEVEES WITHOUT LIQUEFIED SOILS

7.40 General

It has long been recognized that the soft soils comprising the marginal levees and their foundations may be susceptible to unacceptably large earthquake-induced deformations even without the development of liquefaction. One such case history may be the failure of the Southern Pacific Railroad embankment in the Suisun Marsh following the 1906 San Francisco Earthquake (see Section 5.64). To examine this possibility, pseudodynamic slope stability analyses were performed and the results were employed with the Makdisi and Seed (1978) simplified method of deformation analysis.

7.41 Makdisi-Seed Method

The Makdisi-Seed method of deformation analysis is a simplified method based on a variation of Newmark's sliding block double integration method. This method employs a pseudodynamic slope stability analysis to determine a yield acceleration, k_y, at which the slope begins to move. This yield acceleration is then compared to the average peak acceleration, k_{max}, induced with the sliding mass during the earthquake. If the yield acceleration is greater than the average peak acceleration developed by the earthquake shaking, little or no movement will take place. If the reverse is true, then every time an acceleration pulse exceeds the yield acceleration, the mass will start to yield and deform.

The amount of movement during each pulse will depend on how much and for how long the acceleration level exceeds the yield value. To calculate the amount of movement, Makdisi and Seed (1978) performed double integration calculations for a variety of earthquake accelerograms and embankment responses. This study resulted in curves to predict slope movements based on the ratio of (k_y/k_{max}) and the magnitude of the earthquake. These curves are shown in Figure 7-7.

7.42 Determination of Yield Acceleration

As discussed previously, the yield acceleration, k_y, is the seismic coefficient that produces a factor of safety of unity in a pseudodynamic slope stability analysis. Program PCSLOPE was used to obtain the minimum factor of safety for a range of seismic coefficients. The same stability model was employed for the deformation analyses as was used for the liquefaction studies, except that all materials were assigned their static shear strengths (see Figure 7-4). No provision was made for possible strength reductions in the soft clayey and peaty foundation layers. These calculations lead to the determination of a k_y value of 0.055g.

7.43 Determination of Average Peak Acceleration in Potential Slide Mass

In order to estimate the level of earthquake-induced deformation, the Makdisi-Seed method requires an estimate of the average peak acceleration within the potential slide mass, k_{max}. To develop this estimate the following assumptions were made:

1. The average peak acceleration within the potential slide mass, k_{max}, was assumed to be equal to approximately two-thirds the value of the peak acceleration of the levee crown. This assumption is based on several previous studies including Seed, et al. (1985).

2. The peak acceleration of the levee crown was assumed to be equal to a function of the peak acceleration of the rock beneath the soil profile. This would be analogous to using an amplification factor for the base peak acceleration. For purposes of this preliminary evaluation, two amplification values were used: 1.0 and 1.6. These values were adopted because they represent the general range obtained from the dynamic response analyses performed in Chapter 6.

7.44 Estimates of Earthquake-Induced Deformation

Presented in Table 7-1 are results of deformation computations calculated for a range of potential earthquake magnitudes and bedrock accelerations. In general, the results

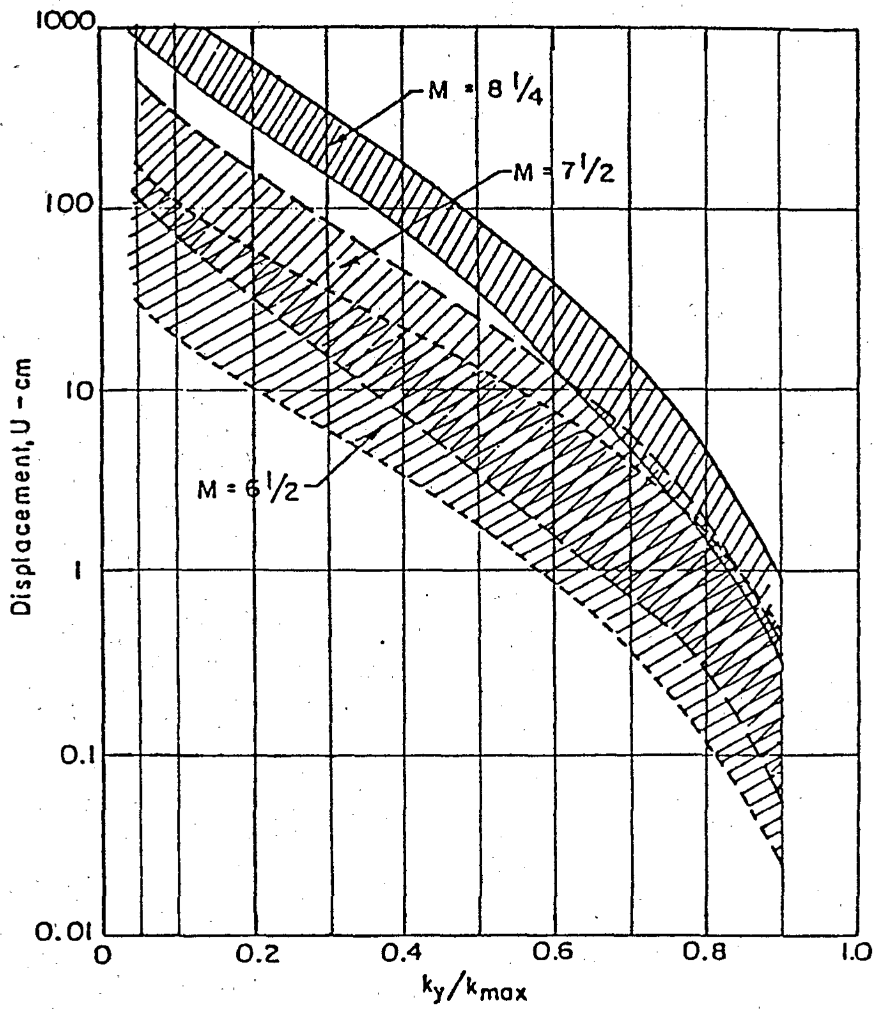


Figure 7-7: Variation of Permanent Displacement With Yield Acceleration (from Makdisi and Seed, 1978)

TABLE 7-1: ESTIMATES OF EARTHQUAKE-INDUCED DEFORMATION

EARTHQUAKE MAGNITUDE	BEDROCK PEAK ACCELERATION (g)	AMPLIFICATION FACTOR	LEVEE CROWN PEAK ACCELERATION (g)	k_{max} (g)	k_y (g)	k_y/k_{max}	EARTHQUAKE-INDUCED DEFORMATION (feet)
6.5	0.10	1.0	0.10	0.067	0.055	0.83	< 0.1
	0.15	1.0	0.15	0.100	0.055	0.55	0.1 - 0.3
	0.20	1.0	0.20	0.133	0.055	0.41	0.1 - 0.7
6.5	0.10	1.6	0.16	0.107	0.055	0.52	0.1 - 0.4
	0.15	1.6	0.24	0.160	0.055	0.34	0.1 - 1.0
	0.20	1.6	0.32	0.213	0.055	0.26	0.2 - 1.4
7.5	0.10	1.0	0.10	0.067	0.055	0.83	< 0.1
	0.15	1.0	0.15	0.100	0.055	0.55	0.1 - 0.6
	0.20	1.0	0.20	0.133	0.055	0.41	0.2 - 1.4
7.5	0.10	1.6	0.16	0.107	0.055	0.52	0.1 - 0.7
	0.15	1.6	0.24	0.160	0.055	0.34	0.4 - 2.1
	0.20	1.6	0.32	0.213	0.055	0.26	0.6 - 3.4
8.5	0.10	1.0	0.10	0.067	0.055	0.83	< 0.1
	0.15	1.0	0.15	0.100	0.055	0.55	0.7 - 1.9
	0.20	1.0	0.20	0.133	0.055	0.41	2.3 - 5.4
8.5	0.10	1.6	0.16	0.107	0.055	0.52	0.9 - 2.4
	0.15	1.6	0.24	0.160	0.055	0.34	3.7 - 8.8
	0.20	1.6	0.32	0.213	0.055	0.26	6.5 - 15.0

show that moderate earthquakes with magnitudes of about 7 or less would result in deformations of less than 3 feet. This overall deformation level might correspond to settlements of only about 1.5 to 2 feet, based on the performance of previous embankments (e.g., Seed, et al., 1973; De Alba, et al., 1985). Accordingly, these deformations would probably not result in failure. This last conclusion is based on recent observations of landslide slips at Sherman, Twitchell, and Tyler Islands where slumping induced simply by static loading on the order of 1 to 3 feet was observed without failure of the levee.

The calculations for a great earthquake with a magnitude equal to 8 or higher, however, would induce deformations ranging as high as 4 to 15 feet if peak bedrock accelerations exceeded a value of approximately 0.15-0.20g. This is in good accord with the estimate of 0.18g peak bedrock acceleration at the Suisun Marsh embankment failure during the 1906 San Francisco Earthquake (M = 8+). Thus, it would seem that a great earthquake would be required to induce a non-liquefaction deformation failure. Because the only fault capable of such an event is the San Andreas Fault, located several miles to the west, it would appear that only areas of the western Delta are susceptible to failure by this mechanism (see Figure 4-2).

7.45 Limitations of Makdisi-Seed Method

The Makdisi-Seed method is a very useful tool, but suffers from the following weaknesses:

1. There has to be some amount of deformation and/or shear strain developed in order to mobilize the high shear strengths needed to resist strong earthquake shaking. The Makdisi-Seed method does not account for this necessary deformation.

2. The Makdisi-Seed method assumes that deformations take place on individual planes rather than being distributed within a zone of an embankment. Analyses performed by Khamenehpour (1983) indicate that this assumption may lead to somewhat conservative evaluations of actual performance.

3. Potential errors may be introduced by decoupling the dynamic response portion of the analysis from the deformation portion of the analysis. Some investigators have stated that once a potential sliding surface begins yielding, it can no longer transmit accelerations higher than the yield acceleration of that particular sliding mass. This would lead to lower

average accelerations than those calculated using the procedures employed in this study.

For these reasons, and the uncertainties involved in the dynamic response estimates, the Makdisi-Seed method is considered to predict levels of deformation greater than believed to actually occur, and is therefore considered conservative for evaluation purposes.

8. PREDICTION OF PERFORMANCE

8.0 GENERAL

Previous chapters have discussed the difficulties involved in making meaningful predictions of seismic stability for the levees protecting the Sacramento-San Joaquin Delta. With hundreds of miles of levees, variable geometries, variable levee materials, and variable foundations, precise predictions are not feasible. Nevertheless, some insight can be gained by using the available data, together with the results of the preliminary analyses performed in the previous chapters.

8.1 ESTIMATED SUSCEPTIBILITY FOR LEVEE DAMAGE

8.10 Definitions of Susceptibility

As noted above, precise assessments for every reach in the Delta are not feasible with the available information and the serious questions regarding the amplification characteristics of the peaty soils. However, it is possible to provide an estimate of those zones in the Delta that are most likely to experience levee damage. The following criteria were used in delineating potential levee damage susceptibilities:

> High - It is likely that there would be widespread liquefaction of sandy and/or silty levees, probably resulting in sufficient losses of freeboard to cause overtopping and subsequent inundation of the island or tract. Extensive cracking leading to piping failures of the levees is also expected to be common in this area. Old stream channels would probably liquefy and may also lead to stability failures of overlying levees. Levee crown peak accelerations will be at least 0.30g (normalized to a Magnitude 7.5 event). Past reports of earthquake-induced damage indicate that this level of shaking has not occurred in the Delta since the levees were first constructed in the 1870s.
>
> Moderately-High - It is likely that isolated reaches of levees would develop extensive liquefaction and result in significant loss of freeboard. In such areas where levees also have relatively little freeboard and/or limited cross sections, overtopping and piping failures are likely. Old stream channels may also develop significant liquefaction and induce failure in overlying levees. Levee crown peak accelerations will be between 0.18g and 0.30g (normalized to

a Magnitude 7.5 event). Past reports of earthquake-induced damage indicate that this level of shaking has not occurred in the Delta since the levees were first constructed in the 1970s.

Low to Moderate - Liquefaction of levee embankments may occur intermittently. In many locations there may be localized slumping and cracking similar to that which occurs during large floods. Levee failure may result if repairs are not made immediately. Levee crown peak accelerations will be between 0.10g and 0.18g (normalized to a Magnitude 7.5 event). Past reports of earthquake-induced damage indicate that the lower bound of this shaking level may have been experienced in portions of the Delta since the construction of the levees.

Low - Locations of liquefaction within levees are sparse and difficult to detect. Minor cracking and slumping will be reported. However, it will be difficult to ascertain whether it was pre-existing or a result of the earthquake. Some pre-earthquake seeps may change flow rates, or may even stop flowing. No major repairs would be expected as a result of the earthquake shaking. Levee crown peak accelerations will be less than 0.10g (normalized to a Magnitude 7.5 event). Past reports of earthquake-induced damage indicate that this level of shaking has already been experienced in portions of the Delta without significant levee damage.

The qualitative ratings were developed based on the past performance of the levees during earthquake shaking and on the liquefaction assessments made for levees as detailed in Chapter 7.

8.11 Ground Motion Amplification Factors Assumed for Estimates of Susceptibility of Levee Failure

Previous chapters have detailed that the largest unknown with respect to making seismic stability estimates for potential earthquake-induced levee failures is the unknown amplification characteristics of the peaty soils. In order to make even qualitative estimates of failure susceptibility, it is necessary to have at least a rough estimate of the ground motion amplification in order to determine accelerations of the levee. Dynamic response analyses presented in Chapter 6 indicated a potential range in amplification factors of between 0.4 and 2.8. For the purposes of this preliminary analysis, this large potential range in amplification was considered too extreme. Instead a range of amplification factors between 1.0 and 1.6 was assumed. This range included more than half of the values calculated in the response analyses and was considered appropriate for the following reasons:

- The very small amplification factors of 0.4 were obtained using properties developed for unconsolidated peaty soil. However, materials beneath the levee are significantly consolidated by the weight of the levee and often are intermixed with the mineral soils of the natural levee system preexisting reclamation. Thus, a minimum amplification factor of unity seems appropriate for the low estimate.

- The very large amplification factors of 2.8 or higher are not consistent with the good performance of levees in the southwestern Delta during the 1989 Loma Prieta Earthquake. During this earthquake, such levels of amplification were observed on the margins of San Francisco Bay and are thought to have been responsible for the damage which occurred at the Marina District, Treasure Island, and the Cypress Freeway. However, the southwestern portion of the Delta is located approximately the same distance away from the Loma Prieta epicenter as were these sites (see Figure 8-1). Accordingly, amplification factors higher than 2 would seem to be unlikely. If amplification factors were this high, then noticeable levee damage would have been expected to have occurred near Clifton Court, Victoria Island, and Byron Tract.

- The only available test data of Delta peats was obtained from Bouldin Island. These data indicated modulus reduction and damping curves similar to those developed by Sun, et al. (1988), for San Francisco Bay Mud. When Bay Mud properties were used in the dynamic response analyses presented in Chapter 6, the average amplification factor was about 1.6. Consequently, the use of a 1.6 amplification factor for the upper range seems appropriate.

8.12 Locations of Levees Susceptible to Earthquake-induced Damage

Estimated locations for levee damage susceptibility were determined using the above criteria for three sets of earthquake loading conditions:

1. Exposure period = 30 years
   Probability of non-exceedance = 50 percent
   Amplification Ratio = 1.0

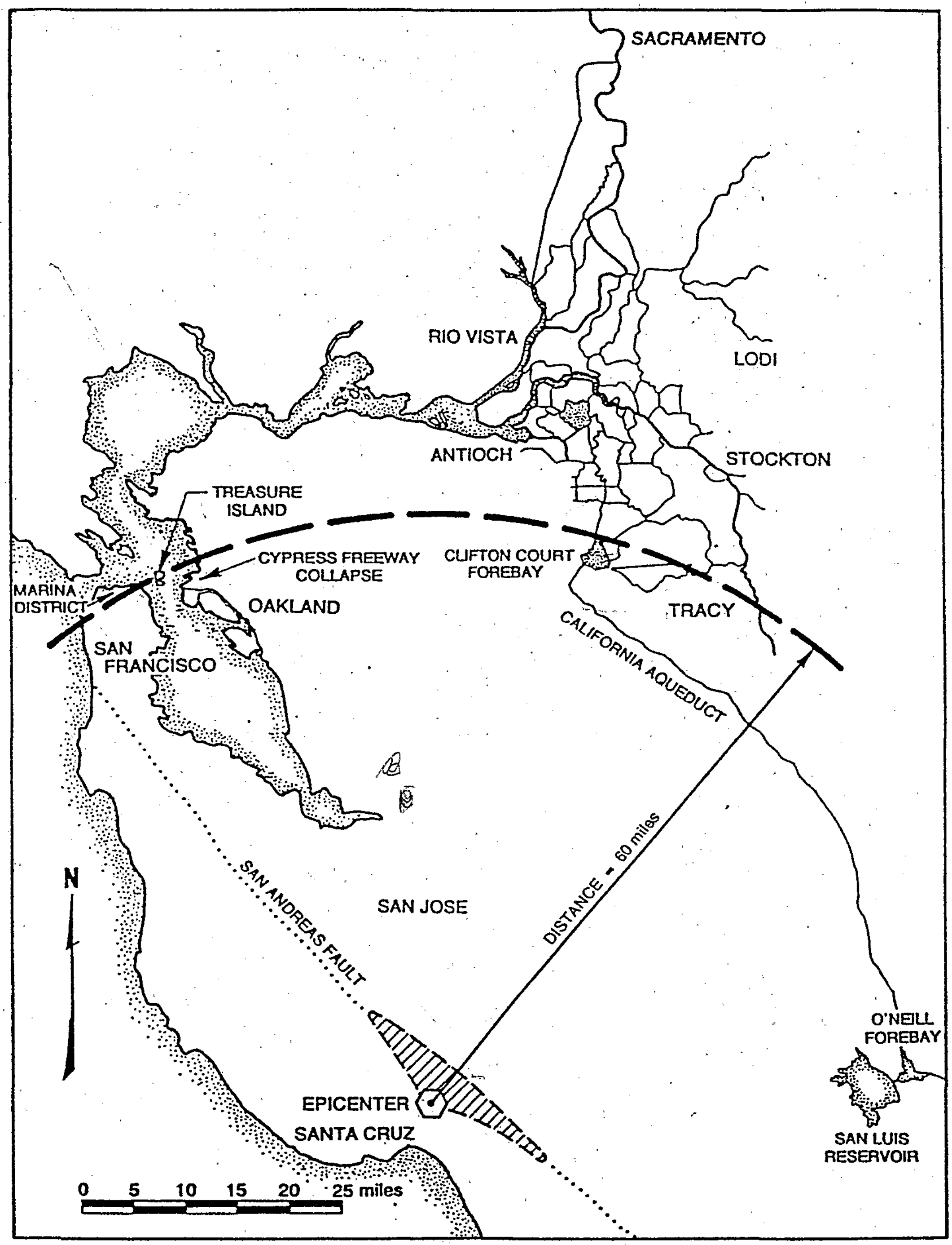


Figure 8-1: Location of 1989 Loma Prieta Fault Rupture and Epicenter

2. Exposure period = 30 years
   Probability of non-exceedance = 50 percent
   Amplification Ratio = 1.6

3. Exposure period = 50 years
   Probability of non-exceedance = 90 percent
   Amplification Ratio = 1.0

Estimated zones of levee damage susceptibilities are shown in Figures 8-2 through 8-4. The zones plotted are not intended to imply that all levee reaches in the zones have the same susceptibilities. Rather, it is expected that at least some portions of each levee reach will have sufficiently liquefiable material to result in the susceptibility identified.

The estimates indicate that only the westernmost portions of the Delta have a moderately high probability of experiencing levee damage within 30 years if an amplification factor of unity is assumed (see Figure 8-2). However, if the amplification factor was increased to 1.6, or if the exposure period was increased to 50 years with a probability of non-exceedance of 90 percent, the entire western half of the Delta is shown to have a moderately high susceptibility to levee damage (see Figures 8-3 and 8-4).

8.13 Earthquake Scenarios

The zones of damage susceptibility presented in Figures 8-2 through 8-4 present envelopes of susceptibility. It should also be noted that, as for the contours of peak acceleration shown in Chapter 4 for the deterministic method, no one earthquake is expected to necessarily result in as much levee damage as is covered by the size of the zones that are plotted. Rather, a single earthquake will have a damage intensity much more limited in geographic extent that the zones shown. To make this point, two scenario earthquake events were developed. These scenario events postulate the occurrence of a moderate earthquake on two particular faults and develop damage contours using the same criteria established above. The assumptions employed in developing these scenarios are as follows:

1. Moderate-sized earthquakes rather than maximum magnitudes associated with each fault:

   a. Magnitude 6.8 event on southern segment of Hayward Fault.

   b. Magnitude 6.3 event on Green Valley Fault.

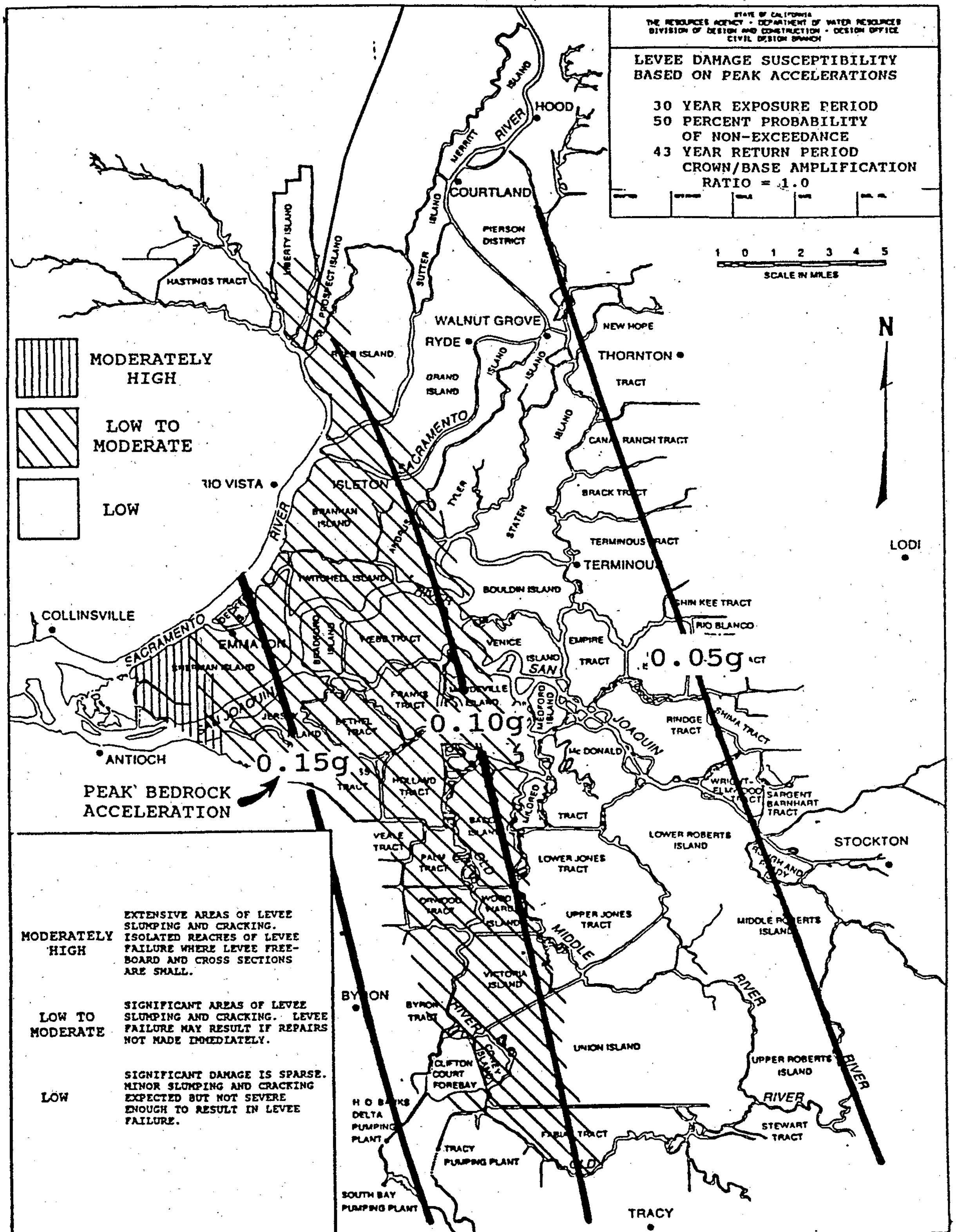

LEVEE DAMAGE SUSCEPTIBILITY
BASED ON PEAK ACCELERATIONS
30 YEAR EXPOSURE PERIOD
50 PERCENT PROBABILITY
OF NON-EXCEEDANCE
43 YEAR RETURN PERIOD
CROWN/BASE AMPLIFICATION
RATIO = 1.0
SCALE IN MILES
N
MODERATELY HIGH
LOW TO MODERATE
LOW
0.05g
0.10g
0.15g
PEAK BEDROCK ACCELERATION
HOOD
COURTLAND
WALNUT GROVE
RYDE
THORNTON
RIO VISTA
ISLETON
TERMINOUS
LODI
COLLINSVILLE
EMMATON
ANTIOCH
STOCKTON
BYRON
TRACY
SACRAMENTO
RIVER
SAN
JOAQUIN
MIDDLE
MODERATELY HIGH
EXTENSIVE AREAS OF LEVEE SLUMPING AND CRACKING. ISOLATED REACHES OF LEVEE FAILURE WHERE LEVEE FREE-BOARD AND CROSS SECTIONS ARE SMALL.
LOW TO MODERATE
SIGNIFICANT AREAS OF LEVEE SLUMPING AND CRACKING. LEVEE FAILURE MAY RESULT IF REPAIRS NOT MADE IMMEDIATELY.
LOW
SIGNIFICANT DAMAGE IS SPARSE. MINOR SLUMPING AND CRACKING EXPECTED BUT NOT SEVERE ENOUGH TO RESULT IN LEVEE FAILURE.

FIGURE 8-2

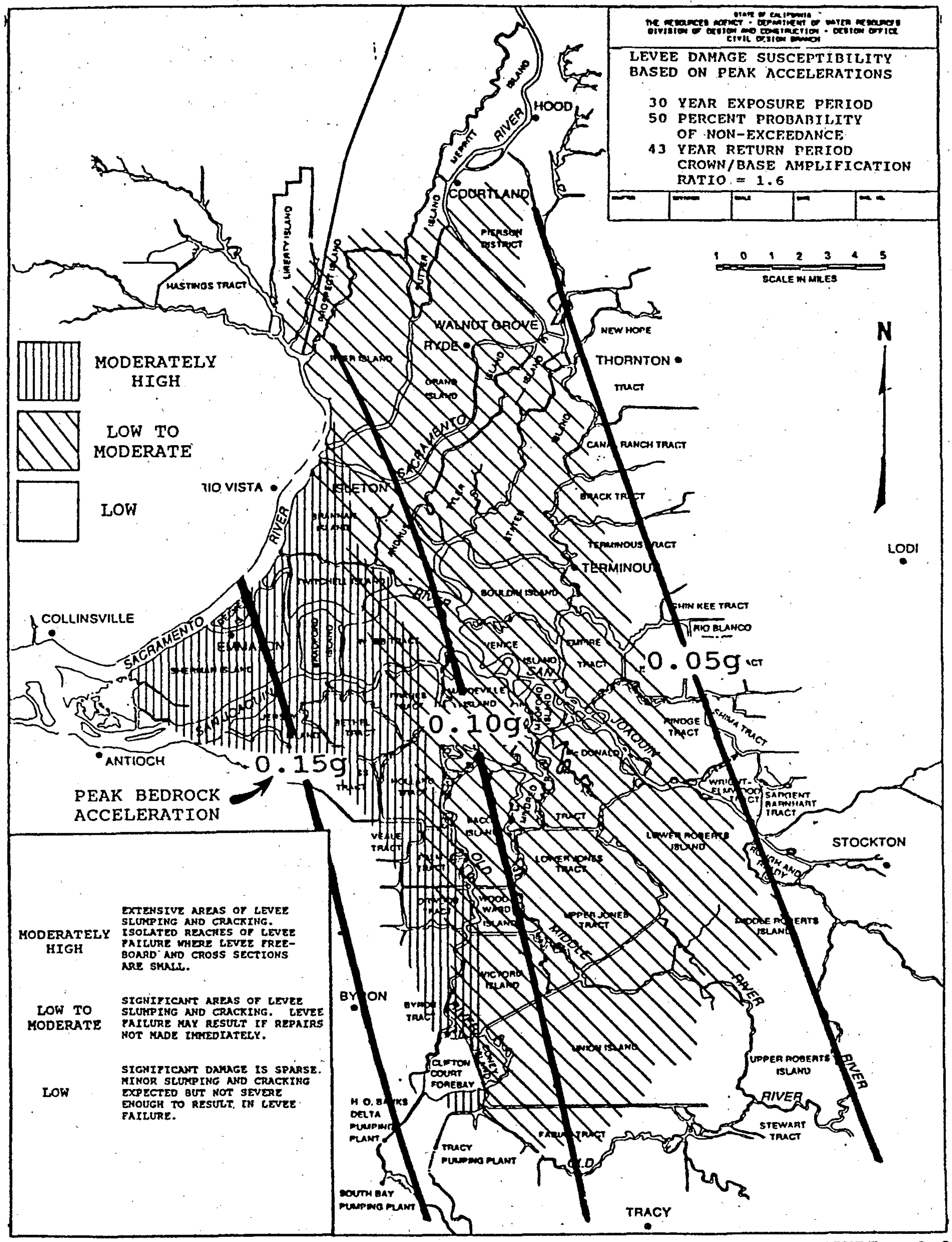

STATE OF CALIFORNIA
THE RESOURCES AGENCY · DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION · DESIGN OFFICE
CIVIL DESIGN BRANCH
LEVEE DAMAGE SUSCEPTIBILITY
BASED ON PEAK ACCELERATIONS
30 YEAR EXPOSURE PERIOD
50 PERCENT PROBABILITY
OF NON-EXCEEDANCE
43 YEAR RETURN PERIOD
CROWN/BASE AMPLIFICATION
RATIO = 1.6
SCALE IN MILES
N
MODERATELY HIGH
LOW TO MODERATE
LOW
0.05g
0.10g
0.15g
PEAK BEDROCK ACCELERATION
HOOD
COURTLAND
WALNUT GROVE
RYDE
NEW HOPE
THORNTON
RIO VISTA
ISLETON
TERMINOUS
LODI
COLLINSVILLE
ANTIOCH
STOCKTON
BYRON
TRACY
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
PIERSON DISTRICT
GRAND ISLAND
SACRAMENTO
RIVER
SAN JOAQUIN
OLD
MIDDLE
BOULDIN ISLAND
RIO BLANCO
UNION ISLAND
UPPER ROBERTS ISLAND
LOWER ROBERTS ISLAND
MIDDLE ROBERTS ISLAND
STEWART TRACT
FABIAN TRACT
CLIFTON COURT FOREBAY
TRACY PUMPING PLANT
H O. BANKS DELTA PUMPING PLANT
SOUTH BAY PUMPING PLANT
MODERATELY HIGH — EXTENSIVE AREAS OF LEVEE SLUMPING AND CRACKING. ISOLATED REACHES OF LEVEE FAILURE WHERE LEVEE FREE-BOARD AND CROSS SECTIONS ARE SMALL.
LOW TO MODERATE — SIGNIFICANT AREAS OF LEVEE SLUMPING AND CRACKING. LEVEE FAILURE MAY RESULT IF REPAIRS NOT MADE IMMEDIATELY.
LOW — SIGNIFICANT DAMAGE IS SPARSE. MINOR SLUMPING AND CRACKING EXPECTED BUT NOT SEVERE ENOUGH TO RESULT IN LEVEE FAILURE.

FIGURE 8-3

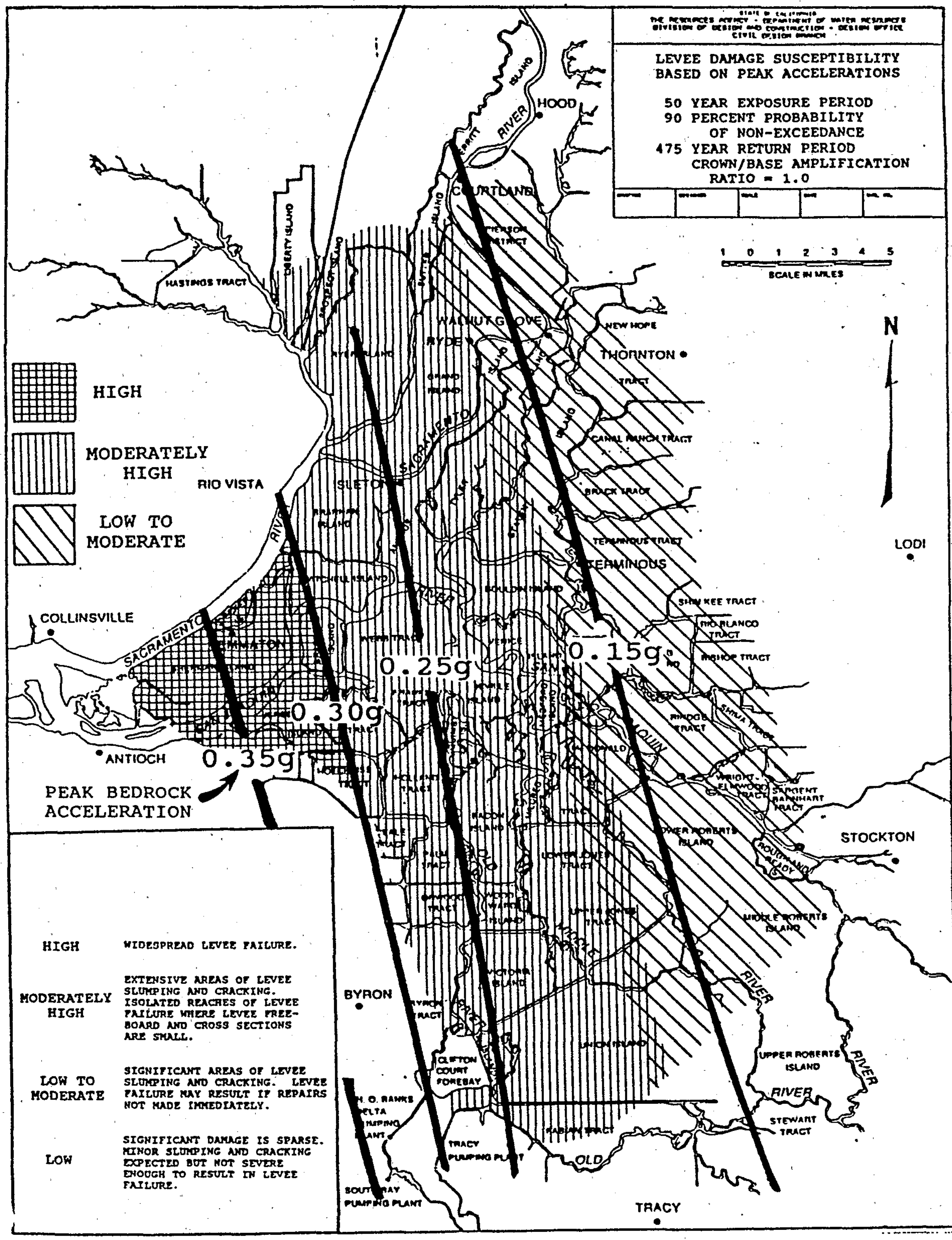

LEVEE DAMAGE SUSCEPTIBILITY
BASED ON PEAK ACCELERATIONS
50 YEAR EXPOSURE PERIOD
90 PERCENT PROBABILITY
OF NON-EXCEEDANCE
475 YEAR RETURN PERIOD
CROWN/BASE AMPLIFICATION
RATIO = 1.0
1 0 1 2 3 4 5
SCALE IN MILES
N
HIGH
MODERATELY HIGH
LOW TO MODERATE
0.35g
0.30g
0.25g
0.15g
PEAK BEDROCK ACCELERATION
HOOD
COURTLAND
HASTINGS TRACT
WALNUT GROVE
RYDE
NEW HOPE
THORNTON
CANAL RANCH TRACT
RIO VISTA
TERMINOUS
LODI
COLLINSVILLE
ANTIOCH
STOCKTON
BYRON
TRACY
UPPER ROBERTS ISLAND
STEWART TRACT
SACRAMENTO
RIVER
OLD
HIGH
WIDESPREAD LEVEE FAILURE.
MODERATELY HIGH
EXTENSIVE AREAS OF LEVEE SLUMPING AND CRACKING. ISOLATED REACHES OF LEVEE FAILURE WHERE LEVEE FREE-BOARD AND CROSS SECTIONS ARE SMALL.
LOW TO MODERATE
SIGNIFICANT AREAS OF LEVEE SLUMPING AND CRACKING. LEVEE FAILURE MAY RESULT IF REPAIRS NOT MADE IMMEDIATELY.
LOW
SIGNIFICANT DAMAGE IS SPARSE. MINOR SLUMPING AND CRACKING EXPECTED BUT NOT SEVERE ENOUGH TO RESULT IN LEVEE FAILURE.

FIGURE 8-4

2. Upper limit of expected soil amplification factor (i.e., Crown/Base Amplification Factor = 1.6).

The zones of damage susceptibilities developed from these two scenarios are shown in Figures 8-5 and 8-6. As may be observed, the figures show relatively limited zones of low to moderate damage susceptibility in the western areas of the Delta. Thus, one particular earthquake need not devastate major sections of the Delta. However, if the earthquake in question was larger, or located on a closer fault, then higher levels of damage would be postulated.

8.2 UNKNOWNS

The are several unknowns which have significant effects on evaluations of levee stability during earthquake shaking. The unknowns which will have the largest effects on assessments of levee stability during earthquakes are listed below in descending order of importance:

a. Amplification/attenuation characteristics of shallow organic soils.

b. Liquefaction resistance of levee fills.

c. Strength-loss potential in cohesive/organic soils following earthquake shaking.

d. Amplification/attenuation characteristics of deep soil profiles.

e. Liquefaction resistance of foundation soils.

f. Probability of Coast Range - Sierra Nevada Boundary Zone of producing a large magnitude earthquake ($M \geq 6.5$) within the Delta.

8.3 RECOMMENDATIONS FOR FUTURE WORK

The many unknowns regarding the behavior of levees overlying organic soils during moderate earthquake shaking has presented a difficult challenge for investigators in making meaningful assessments of seismic stability. In this preliminary study, a limited set of information was used together with a good measure of judgement to make qualitative assessments. Such studies would be greatly improved if field and laboratory studies were performed to obtain information to reduce the current level of uncertainty.

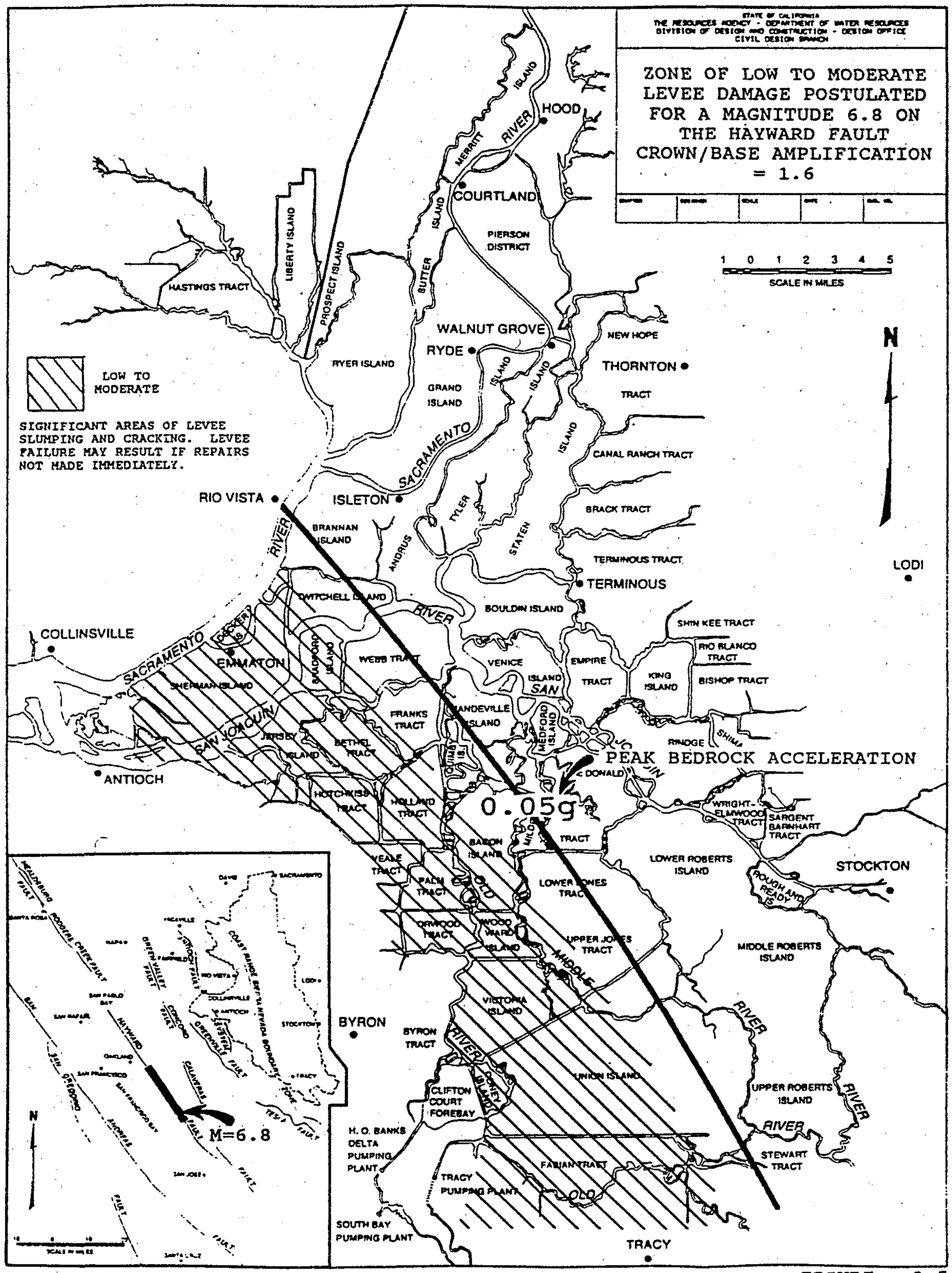

STATE OF CALIFORNIA
THE RESOURCES AGENCY - DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION - DESIGN OFFICE
CIVIL DESIGN BRANCH
ZONE OF LOW TO MODERATE LEVEE DAMAGE POSTULATED FOR A MAGNITUDE 6.8 ON THE HAYWARD FAULT CROWN/BASE AMPLIFICATION = 1.6
SCALE IN MILES
N
LOW TO MODERATE
SIGNIFICANT AREAS OF LEVEE SLUMPING AND CRACKING. LEVEE FAILURE MAY RESULT IF REPAIRS NOT MADE IMMEDIATELY.
HOOD
COURTLAND
PIERSON DISTRICT
HASTINGS TRACT
LIBERTY ISLAND
PROSPECT ISLAND
SUTTER ISLAND
MERRITT ISLAND
RYER ISLAND
WALNUT GROVE
RYDE
GRAND ISLAND
NEW HOPE
THORNTON
CANAL RANCH TRACT
BRACK TRACT
TERMINOUS TRACT
TERMINOUS
LODI
RIO VISTA
ISLETON
BRANNAN ISLAND
ANDRUS
TYLER
STATEN
SACRAMENTO RIVER
TWITCHELL ISLAND
BOULDIN ISLAND
SHIN KEE TRACT
RIO BLANCO TRACT
BISHOP TRACT
KING ISLAND
EMPIRE TRACT
VENICE ISLAND
COLLINSVILLE
EMMATON
SHERMAN ISLAND
BRADFORD ISLAND
WEBB TRACT
FRANKS TRACT
MANDEVILLE ISLAND
MEDFORD ISLAND
SAN JOAQUIN
RINDGE
SHIMA
JERSEY ISLAND
BETHEL TRACT
ANTIOCH
PEAK BEDROCK ACCELERATION
0.05g
MC DONALD
HOTCHKISS TRACT
HOLLAND TRACT
BACON ISLAND
WRIGHT-ELMWOOD TRACT
SARGENT BARNHART TRACT
LOWER ROBERTS ISLAND
STOCKTON
ROUGH AND READY IS.
VEALE TRACT
PALM TRACT
LOWER JONES TRACT
ORWOOD TRACT
WOODWARD ISLAND
UPPER JONES TRACT
MIDDLE ROBERTS ISLAND
MIDDLE RIVER
VICTORIA ISLAND
BYRON
BYRON TRACT
UNION ISLAND
UPPER ROBERTS ISLAND
CLIFTON COURT FOREBAY
H. O. BANKS DELTA PUMPING PLANT
TRACY PUMPING PLANT
FABIAN TRACT
OLD RIVER
STEWART TRACT
SOUTH BAY PUMPING PLANT
TRACY
M=6.8
HAYWARD FAULT
CALAVERAS FAULT
SAN ANDREAS FAULT
SACRAMENTO
STOCKTON
OAKLAND
SAN FRANCISCO
SAN JOSE

FIGURE 8-5

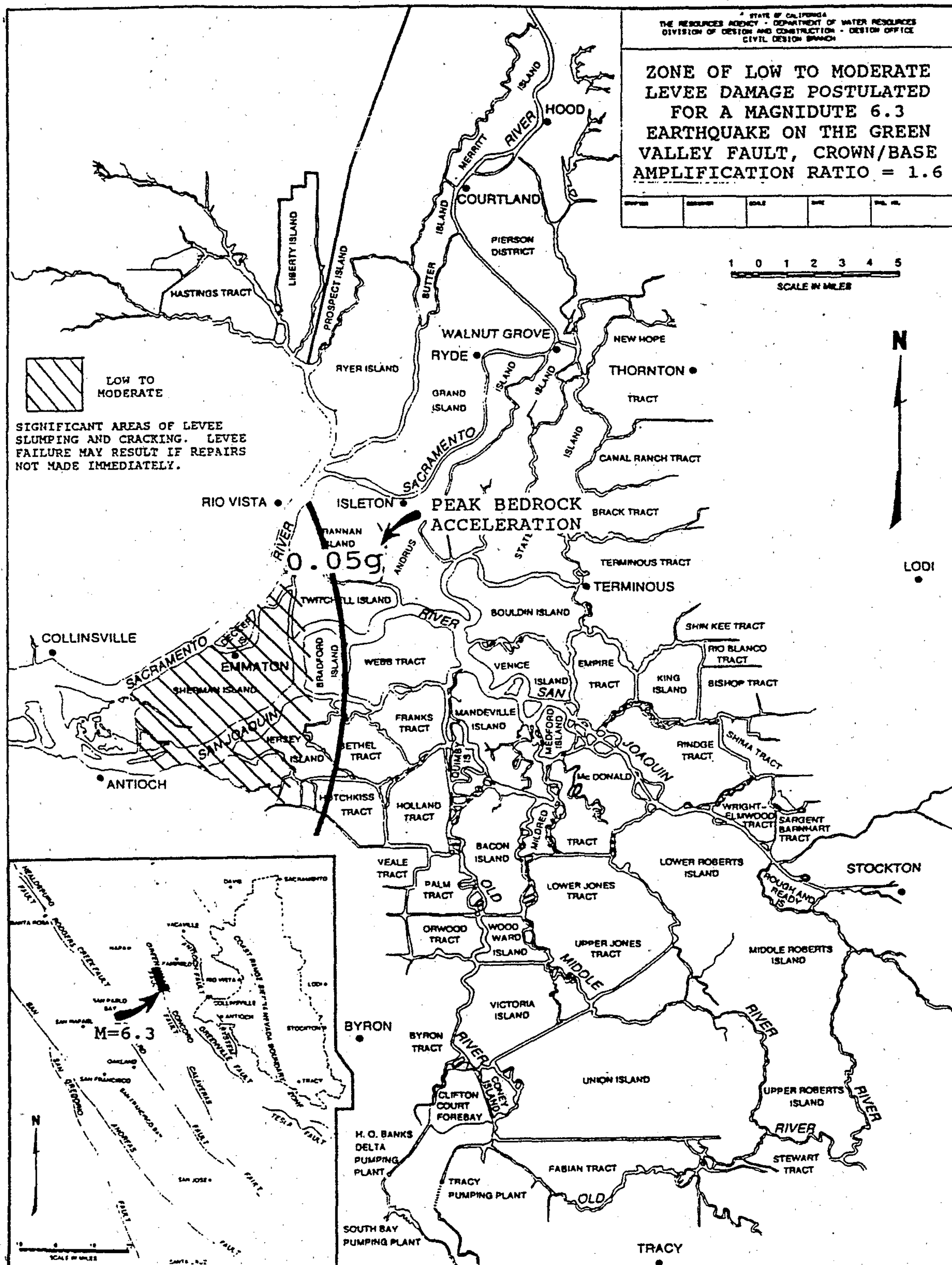

ZONE OF LOW TO MODERATE LEVEE DAMAGE POSTULATED FOR A MAGNIDUTE 6.3 EARTHQUAKE ON THE GREEN VALLEY FAULT, CROWN/BASE AMPLIFICATION RATIO = 1.6
LOW TO MODERATE
SIGNIFICANT AREAS OF LEVEE SLUMPING AND CRACKING. LEVEE FAILURE MAY RESULT IF REPAIRS NOT MADE IMMEDIATELY.
PEAK BEDROCK ACCELERATION
0.05g
M=6.3
SCALE IN MILES
HOOD
COURTLAND
WALNUT GROVE
RYDE
THORNTON
RIO VISTA
ISLETON
LODI
TERMINOUS
COLLINSVILLE
EMMATON
ANTIOCH
STOCKTON
BYRON
TRACY

FIGURE 8-6

If a higher level of confidence is desired for the assessments of levee stability, a significant investigative effort is required. As a minimum effort, the following program is proposed:

1. At three sites, deep boreholes (300 feet plus) should be drilled and sampled. Crosshole shear wave velocity tests should be performed to the full depth. Strong motion accelerometers should be installed at the following depths:

    a. At the contact between soil and rocklike consolidated sediments.

    b. At the base of the organic soil layer.

    c. At the base of the levee fill directly below the crown.

    d. At the crown of the levee.

    e. Out in the freefield beyond the landside levee toe.

2. Develop a geologic model for the deeper soil deposits within the Delta. If possible, it would be desirous to develop depth contours of soils sufficiently consolidated as to be treated as rocklike in dynamic response characteristics.

3. Field and laboratory testing should be performed to better determine the static and dynamic properties of organic soil.

    a. Obtain a number of samples from two different types of peaty soils (e.g., one type could have an organic burn loss of about 50 percent and the other could have an organic burn loss of about 20 percent). The following tests should then be performed:

        o Triaxial compression strength tests.

        o Direct simple shear strength tests.

        o Resonant column tests.

        o Cyclic simple shear tests.

        o Classification tests.

b. The results from sophisticated laboratory tests should be correlated with relatively simplified field tests, such as the cone penetrometer and the field vane shear devices.

c. Low-level vibrations on levee crowns should be recorded to estimate response characteristics of soil profiles.

4. Field and laboratory testing should be performed to better determine the liquefaction potential of levee and foundation soils.

   a. SPT boreholes should be drilled through levee fills and into levee foundations for a 3,000-foot length at approximately 250-foot spacings at three locations. At critical areas, additional boreholes would be performed to characterize continuity. Cone penetration soundings may be substituted for some of the boreholes.

   b. At locations where levees have been built on top of infilled stream channels, additional SPT tests and low-level vibration tests would be performed.

5. Investigations should be made regarding the activity of the Coast Range-Sierra Nevada Boundary Zone and other relevant seismological features.

6. Additional dynamic response, liquefaction, and slope stability analyses should be performed to evaluate levee stability with the data produced from the above studies.

8.4 CLOSING STATEMENT

Regardless of the results developed in these investigations, it is not the intention to repair or design new levees to meet standards developed for earth dams. Rather, it is the purpose of these studies to develop information as to the susceptibility and opportunity for Delta levees to sustain damage during earthquakes. With this information, the degree of risk can be estimated, design criteria can be established, and a rational approach can be pursued in the management of existing and future Delta facilities.

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